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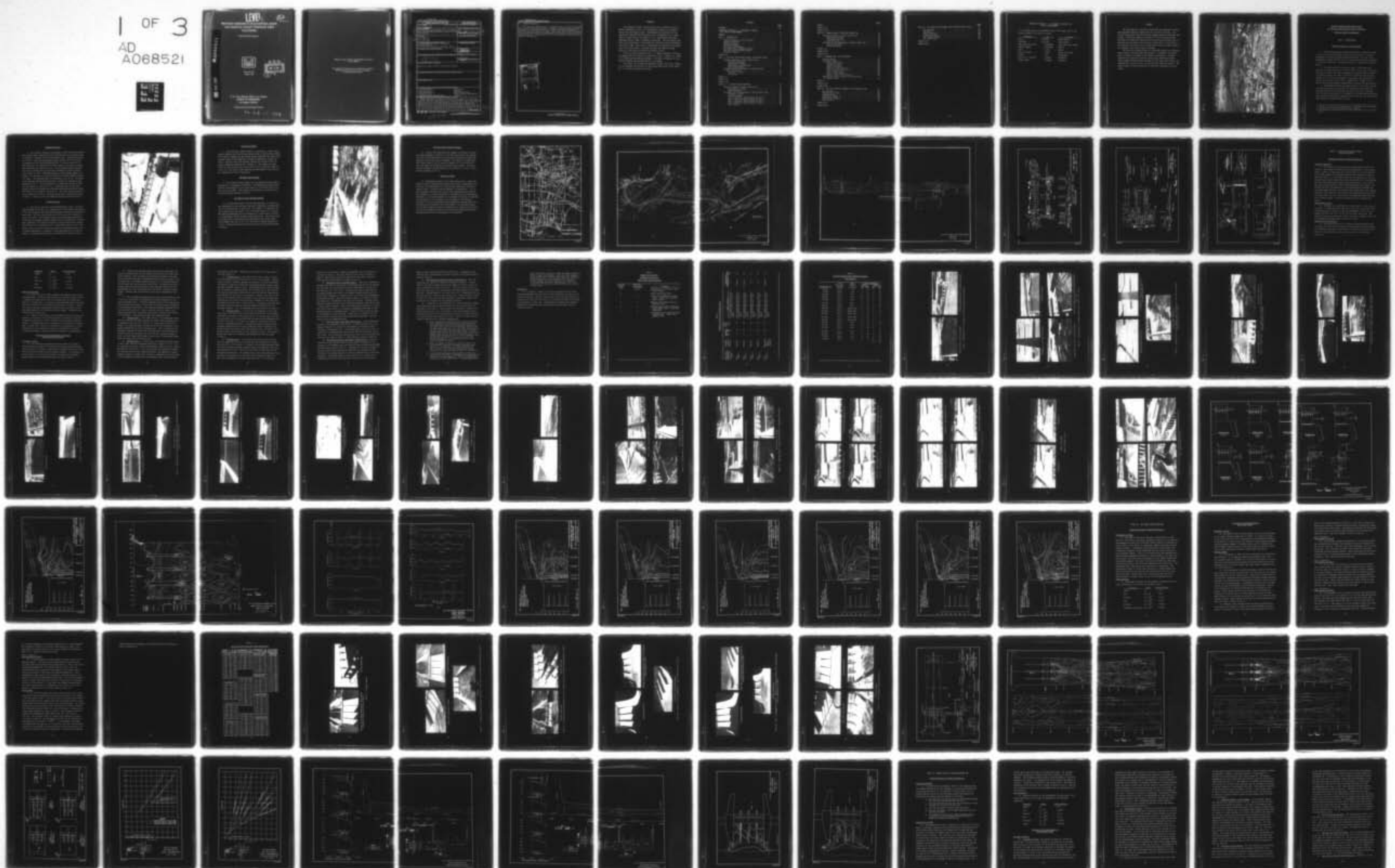

LEVEL IV

(12)

WHITTIER NARROWS FLOOD-CONTROL BASIN
LOS ANGELES COUNTY DRAINAGE AREA
CALIFORNIA

Hydraulic Model Investigation

Report No. 2-112
February 1979

U. S. Army Engineer District, Los Angeles
CORPS OF ENGINEERS
Los Angeles, California

Unclassified

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Drop structures	Spillways
Flow characteristics	Whittier Narrows Dam
Hydraulic models	Whittier Narrows Flood-Control Basin
Los Angeles County Drainage Area	

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The investigation of Whittier Narrows Dam was conducted to verify and supplement hydraulic computations for the initial designs. The various designs were altered to achieve maximum hydraulic efficiency and economy. Particular emphasis was given to the flow characteristics developed in the Whittier Narrows basin from inflow provided by the Rio Hondo and San Grabriel River and the outflow which is controlled by two hydraulic structures, the Rio Hondo outlet works (the main regulating structure) and the San Gabriel spillway. CONT'

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20. ABSTRACT (Continued).

CONT'

The tests were conducted upon six individual models, one being the general model of the Whittier Narrows basin. In general, the overall model functioned as assumed in the design of the project. However, problems were apparent at specific points which needed more detail study, namely the San Gabriel spillway, the Rio Hondo outlet works, the Rosemead Blvd. drop structure, and the San Jose diversion channel. The model studies of these structures are described individually in this report.

FOREWORD

The hydraulic studies reported herein were conducted in the Hydraulic Laboratory of the U. S. Army Engineer District, Los Angeles, during the period 1949 to 1954. Preparation and publication of the report were authorized by the Office, Chief of Engineers, in a letter dated 21 August 1969 to the Director, U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi. The basic model data used in the preparation of this report are available from the Permanent Data Files of WES. This report is submitted, in compliance with regulations of ER 1110-1-8100, to present the data and results of model studies for the Whittier Narrows Flood-Control Basin.

This report was prepared by Mr. D. A. Barela, Hydraulics Section, Los Angeles District, under the supervision of Mr. A. Robles, Jr., Chief of the Hydrology and Hydraulics Branch. COL Hugh G. Robinson, CE, was District Engineer during publication of the report.

The report was reviewed and published by WES. COL John L. Cannon, CE, was Commander and Director of WES during publication of the report; Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acres	4046.856	square metres
acre-feet	1233.482	cubic metres
cubic feet per second	0.02831685	cubic metres per second
degrees (angle)	0.01745329	radians
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	2.54	centimetres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms

SUMMARY

The investigation of Whittier Narrows Dam was conducted to verify and supplement hydraulic computations for the initial designs. The various designs were altered to achieve maximum hydraulic efficiency and economy. Particular emphasis was given to the flow characteristics developed in the Whittier Narrows basin from inflow provided by the Rio Hondo and the San Gabriel River and the outflow which is controlled by two hydraulic structures, the Rio Hondo outlet works (the main regulating structure) and the San Gabriel spillway.

The tests were conducted upon six individual models, one being the general model of the Whittier Narrows basin. In general, the overall model functioned as assumed in the design of the project. However, problems were apparent at specific points which needed more detail study, namely the San Gabriel spillway, the Rio Hondo outlet works, the Rosemead Blvd. drop structure, and the San Jose diversion channel. The model studies of these structures are described individually in this report.

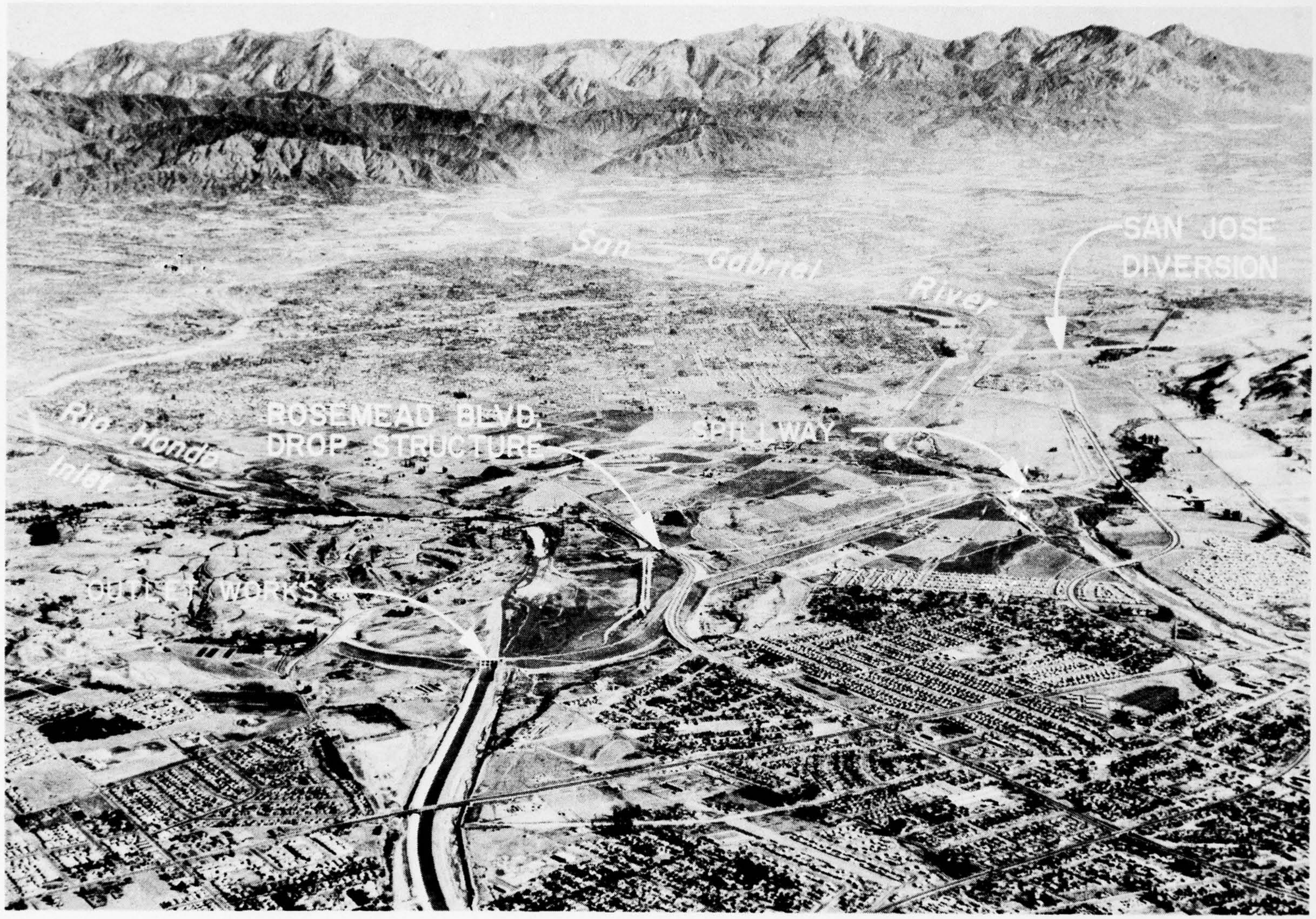

Figure 1. The prototype

WHITTIER NARROWS FLOOD-CONTROL BASIN

LOS ANGELES COUNTY DRAINAGE AREA, CALIFORNIA

Hydraulic Model Investigation

PART I: INTRODUCTION

Pertinent Feature of the Prototype

1. The Whittier Narrows Flood-Control Basin is centrally located on the south side of San Gabriel Valley in the vicinity of a natural topographic constriction known as Whittier Narrows. The site is approximately 3 miles* south of the city of El Monte and 3 miles northwest of the city of Whittier. The general location of the basin is shown in Plate 1.

2. The flood-control basin has an area of 2,865 acres and a capacity of 36,160 acre-feet at el 229.0 ft**. At the top of the earth-fill dam (el 239.0), the basin encompasses 3,641 acres with a capacity of 65,890 acre-feet; the crest of the dam is 16,690 ft long and has a height of 56 ft above the streambed. The general plan and profile of the dam are shown in Plates 2 and 3, respectively. Figure 1 is an aerial view looking upstream showing Whittier Narrows Dam and Reservoir.

3. Appurtenant features discussed briefly in this report include the spillway and outlet structures, the flood-flow channel and drop structure between the Rio Hondo and San Gabriel River just upstream from the dam, short reaches of the channels of Rio Hondo and San Gabriel River within and immediately upstream of the reservoir, and the San Jose Creek diversion channel (Plates 2 and 3).

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page ix.

** Elevations (el) cited herein are in feet referred to mean sea level (msl).

Spillway Structure

4. The spillway structure, consisting of nine 50-ft-wide by 29-ft-high tainter gates, separated by six 8-ft-wide and two 16-ft-wide piers, is located to discharge into the San Gabriel River. The gate sills are at el 200.0. The spillway approach channel consists of a 530-ft-wide rectangular section extending upstream 60.5 ft from the piers of the gate structure. The upstream end of the channel walls diverge as quadrant wing walls on a radius of 35 ft. The discharge through the gates flows into a 538-ft-wide by 200-ft-long rectangular concrete channel with a slope of 0.030. The downstream end of the spillway channel terminates in a dentated sill with 8-ft-high by 5-ft-wide dentates, spaced 5 ft apart. The upstream slope of the dentates is 1V on 2H. The height of the spillway channel walls required to confine the spillway design discharge is 21 ft. The downstream end of these walls diverge as quadrant wing walls on a radius of 75 ft. Downstream from the spillway channel, spillway discharges are confined between grouted-stone, diverging levees for a distance of about 500 ft. The right levee continues on for an additional 1,250 ft. The plan and profile of spillway are shown in Plate 4. Figure 2 shows the completed prototype spillway section.

Outlet Structure

5. The outlet structure, located approximately 2,000 ft from the right abutment of the dam, will discharge into the Rio Hondo and consists of four 30-ft-wide by 19-ft-high gated openings, separated by three 8-ft-wide piers. The four tainter gates are 30 ft wide by 20 ft high with sills at el 184.0. The discharge through the outlet gates flows into a 144-ft-wide rectangular concrete channel, which is connected to a 440-ft-long transition. The transition varies in cross section from a rectangular section at the upstream end to a trapezoidal section, with a base width of 100 ft and 1V-on-2.25H side slopes at the downstream end. The Rio Hondo improvement is connected to the downstream end of the outlet channel. The general plan, profile, and sections are shown in Plate 5.

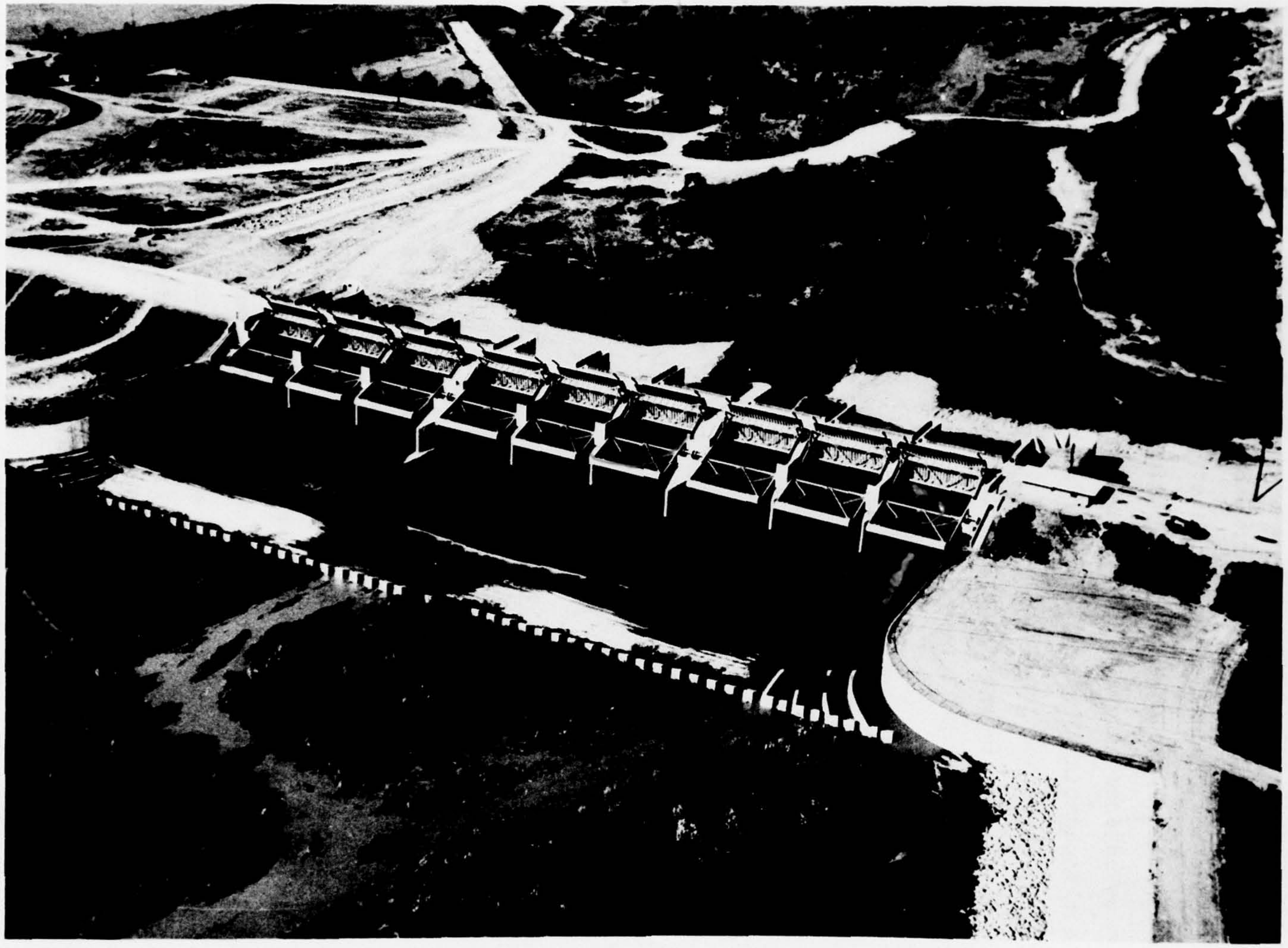


Figure 2. Aerial view looking upstream showing completed spillway section

Flood-Flow Channel

6. The flood-flow channel (Plate 2) is provided to divert excess channel capacity flow from the San Gabriel River to the Rio Hondo outlet. The flood-flow channel and appurtenant intake stabilizer and invert drop structure under the Rosemead Blvd. Bridge are designed to convey low flows to adjacent areas with minimum flooding and to pass 30,000 cfs without inundating Rosemead Blvd. (Figure 3). The channel is trapezoidal in cross section with a base width of 400 ft and side slopes of 1V on 2.5H. The invert slopes upstream and downstream of the structure are 0.002156 and 0.0020, respectively.

Rio Hondo Inlet Channel

7. The Rio Hondo inlet channel into the Whittier Narrows basin is designed for a discharge of 51,000 cfs. The trapezoidal concrete channel has a base width of 150 ft, side slopes of 1V on 2.25H, and an invert slope of 0.00363. Levee heights vary between 14.5 and 16.5 ft to provide a minimum freeboard of 3.0 ft.

San Gabriel River Approach Channel

8. The San Gabriel River from the upstream edge of the basin to the confluence with the San Jose diversion is improved to provide capacity for a design inflow of 99,000 cfs. The channel is trapezoidal in cross section with base width of 450 ft and side slopes of 1V on 2.3H. The channel downstream from the confluence has a base width of 500 ft, is soft-bottomed, and has grouted stone-faced levees. The invert slope is 0.003052 and is stabilized with grouted stone and derrick stone. Levee heights vary from 14.0 to 17.5 feet with a minimum freeboard of 3.0 ft. A partial plan and profile and levee cross sections are shown in Plate 6.

Figure 3. San Gabriel River-Rio Hondo Crossover, Whittier Narrows Dam and Reservoir; flow under Rosemead Blvd. Bridge, storm of Jan 1960, discharge 19,000 cfs

San Jose Creek Diversion Channel

9. The San Jose Creek diversion channel is designed to divert the design discharge of 38,000 cfs from San Jose Creek to San Gabriel River at a point upstream of the flood-control basin, thus permitting regulation of San Jose Creek discharges. The channel is trapezoidal with a base width of 150 ft and side slopes of 1V on 2.3H (Plate 6). The side slopes are protected with grouted stone facing. The invert slope is 0.00290. The levee heights vary from 16.0 to 16.5 ft, with a minimum freeboard of 3.0 ft.

Purpose of Study

10. The general purpose of the model studies was to verify the design of Whittier Narrows Flood-Control Basin and its appurtenant features and to provide means for correcting any unfavorable conditions found to exist. The main area and elements of concern were: (a) the gate discharge calibration for various gate combinations and various gate openings, (b) the determination of spillway gate and counterweight clearance, (c) the effectiveness of the quadrant wing walls in improving flow conditions, (d) the number, size, and shape of the end-sill dentates, (e) the flow characteristics through the spillway and outlet structures and the probable scour downstream of the spillway channel, (f) the flow condition through the Rosemead Blvd. Bridge drop structure, and (g) the streambed erosion at the San Jose Creek-San Gabriel River confluence and the scour downstream of the stabilizer.

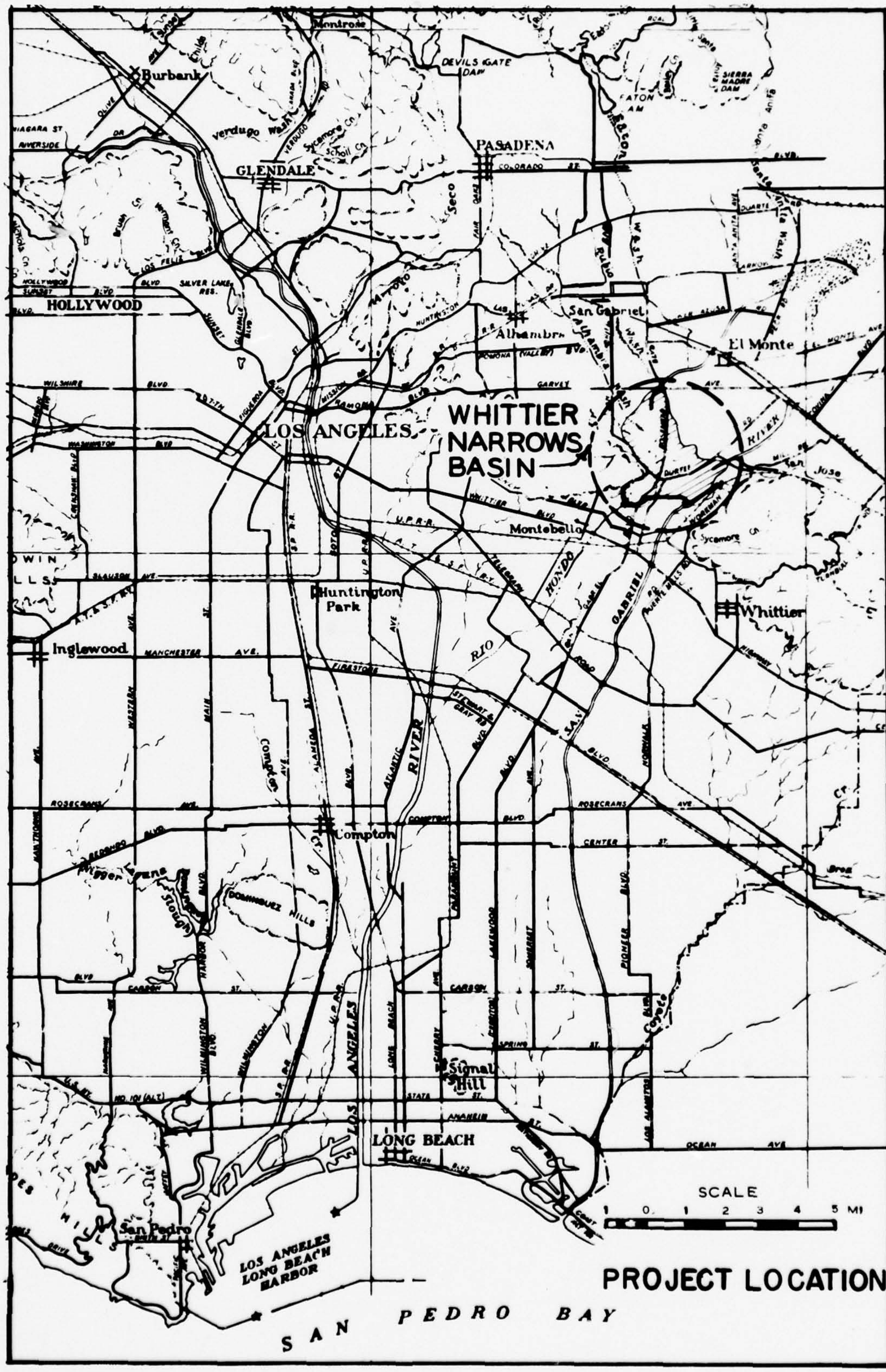
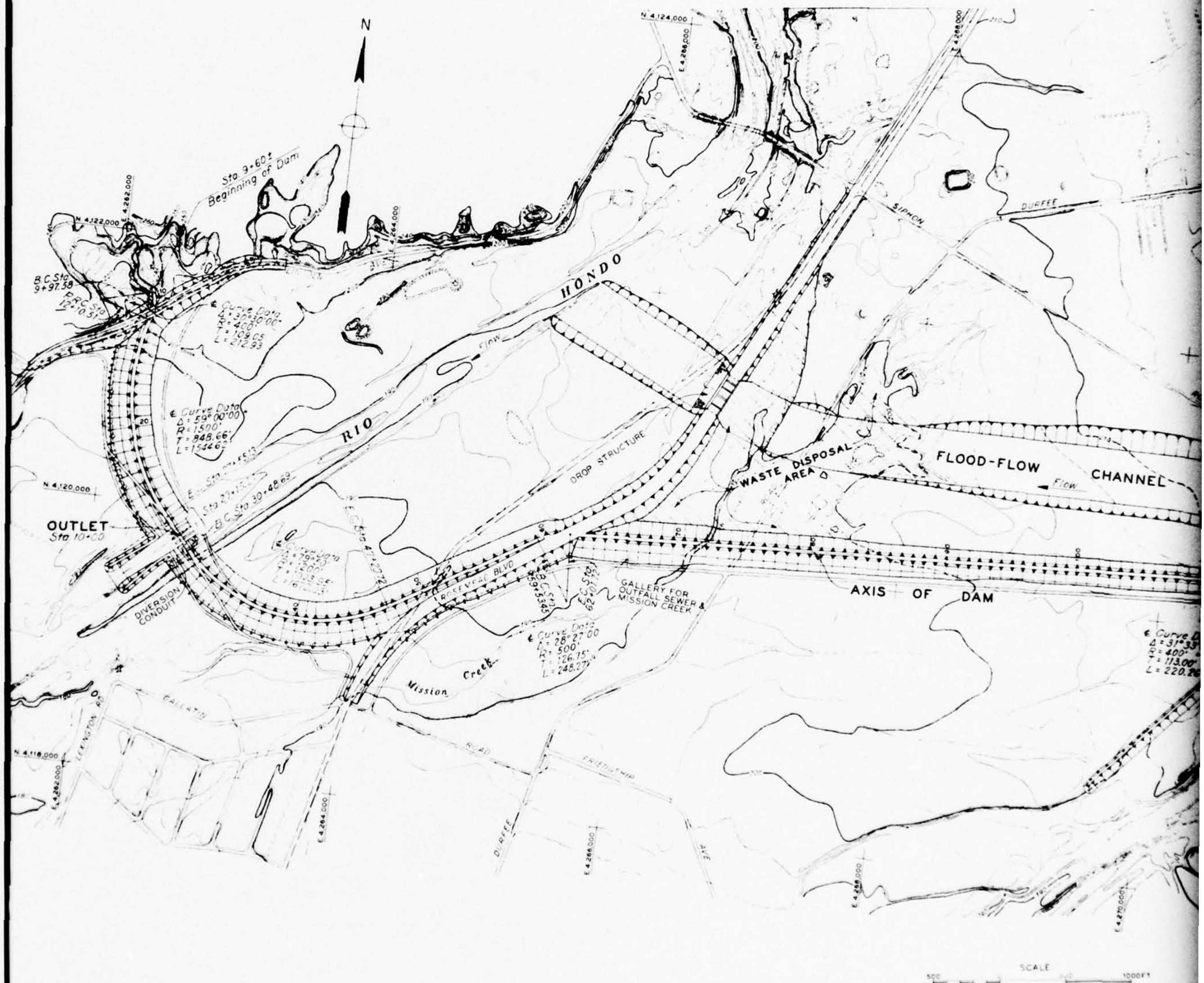
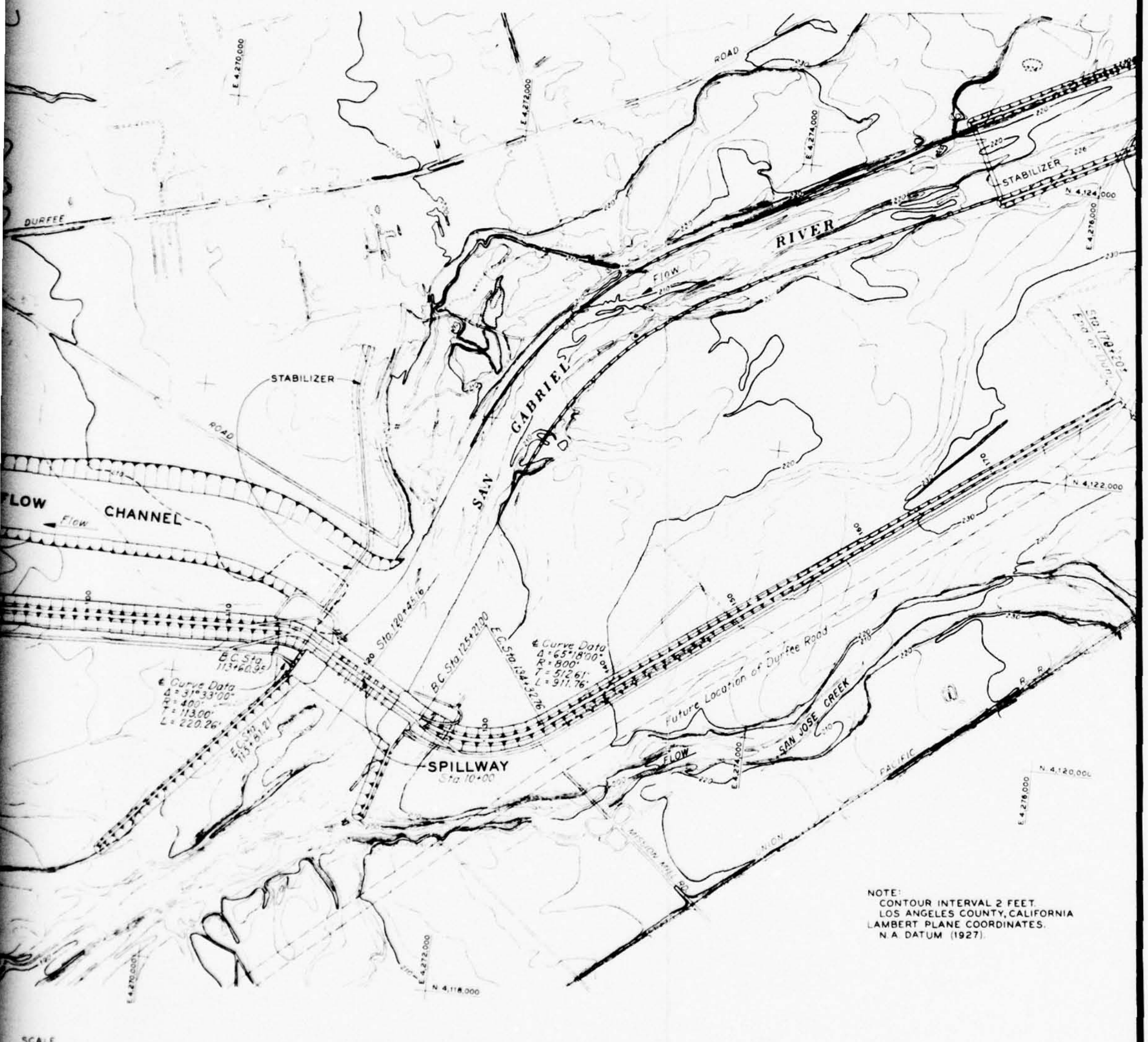

WHITTIER NARROWS BASIN
Burbank
GLENDALE
PASADENA
HOLLYWOOD
LOS ANGELES
San Gabriel
Alhambra
El Monte
Montebello
Huntington Park
Whittier
Inglewood
Compton
Signal Hill
LONG BEACH
San Pedro
LOS ANGELES LONG BEACH HARBOR
SAN PEDRO BAY
SCALE
1 0 1 2 3 4 5 MI
PROJECT LOCATION

PLATE 1


N
Sta 9+60±
Beginning of Dam
N 4,122,000
B.C.Sta 9+97.58
RIO
HONDO
Flow
OUTLET
Sta 10+00
N 4,120,000
DIVERSION CONDUIT
DROP STRUCTURE
WASTE DISPOSAL AREA
FLOOD-FLOW
CHANNEL
Flow
GALLERY FOR OUTFALL SEWER & MISSION CREEK
AXIS OF DAM
Mission Creek
N 4,124,000
N 4,118,000
SIPHON
DURFEE
SCALE
1000 FT

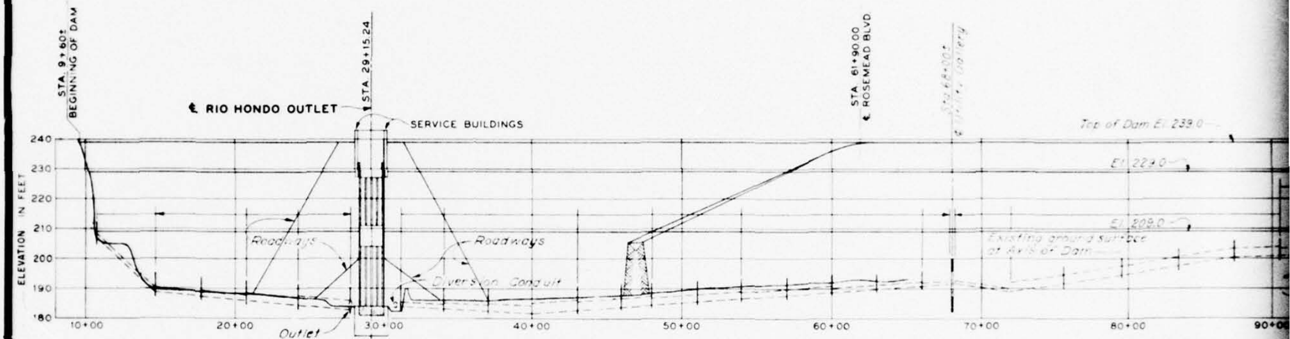
1



WHITTIER NARROWS FLOOD-CONTROL BASIN

GENERAL PLAN

FINAL DESIGN

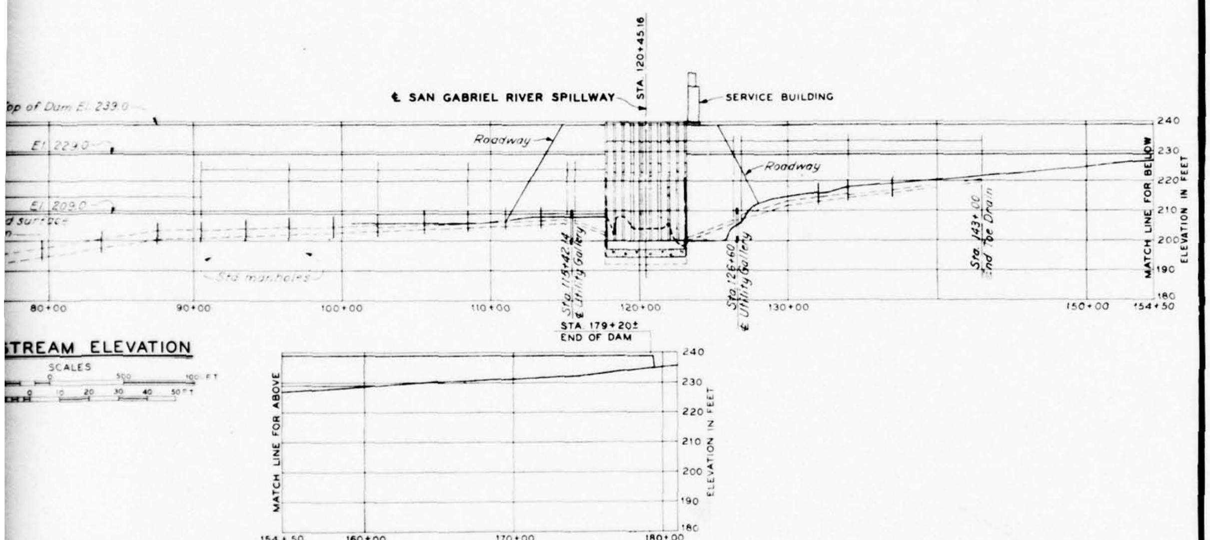


DOWNSTREAM ELEVATION

SCALES

HOR 500 0 500 1000

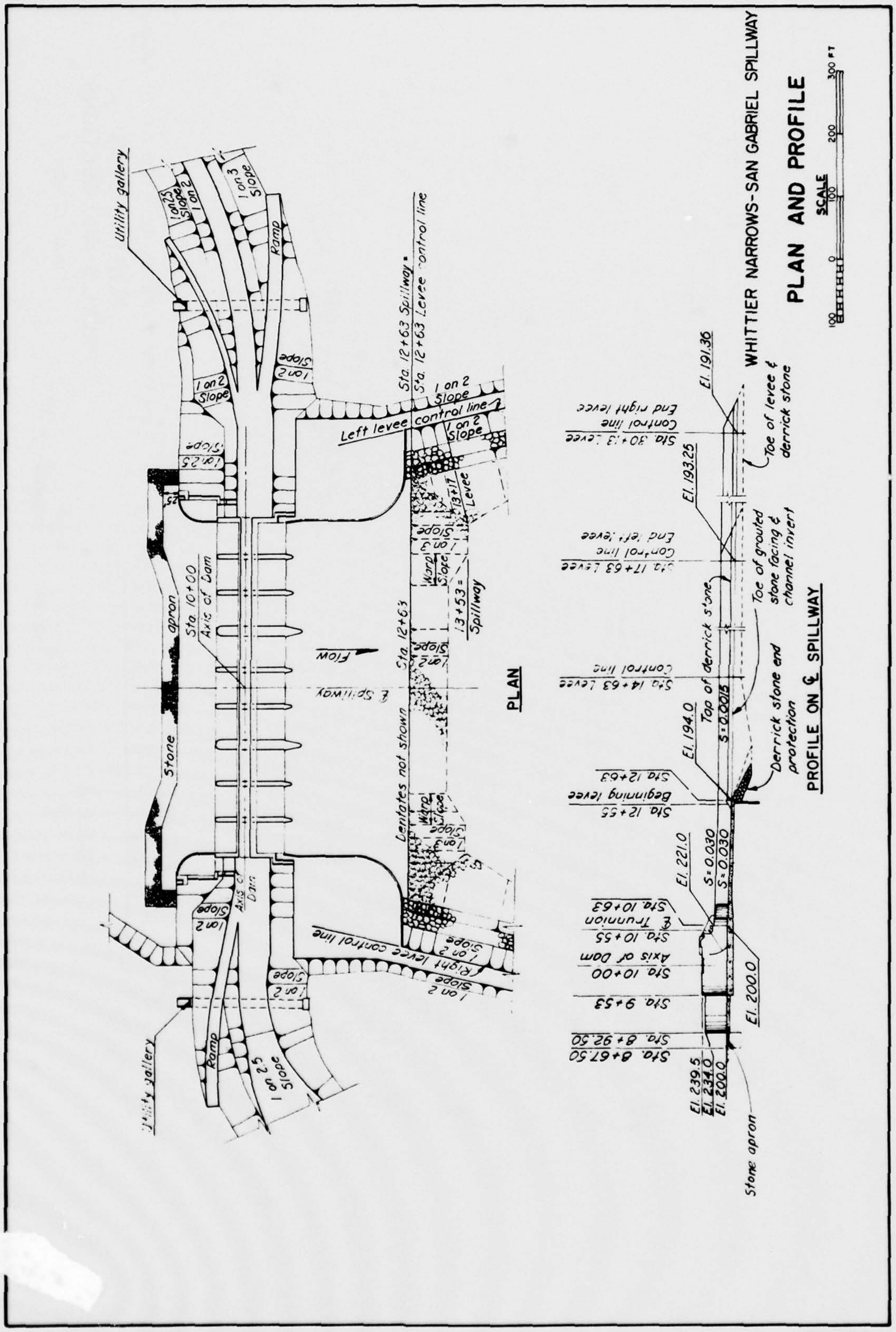
VERT 10 0 10 20 30 40 50 FT



WHITTIER NARROWS FLOOD-CONTROL BASIN

PROFILE

FINAL DESIGN



WHITTIER NARROWS-SAN GABRIEL SPILLWAY
PLAN AND PROFILE
SCALE
100 0 100 200 300 FT
PLAN
PROFILE ON ℄ SPILLWAY
Utility gallery
Stone apron
Sta. 10+00
Axis of Dam
Flow
℄ Spillway
Sta. 12+63
Dentates not shown
Sta. 12+63 Spillway = Sta. 12+63 Levee control line
Left levee control line
Right levee control line
13+53 = Spillway
13+17 Levee
Ramp
1 on 2 Slope
1 on 2.5 Slope
1 on 3 Slope
Warp Slope
Sta. 8+67.50
Sta. 8+92.50
Sta. 9+53
Sta. 10+00 Axis of Dam
Sta. 10+55
℄ Trunnion Sta. 10+63
Sta. 12+55 Beginning levee
Sta. 12+63
Sta. 14+63 Levee Control line
Sta. 17+63 Levee Control line End left levee
Sta. 30+13 Levee Control line End right levee
El. 239.5
El. 234.0
El. 200.0
El. 221.0
S = 0.030
El. 194.0
S = 0.0015
Top of derrick stone
Derrick stone end protection
Toe of grouted stone facing & channel invert
El. 193.25
El. 191.36
Toe of levee & derrick stone
Stone apron

PLATE 4

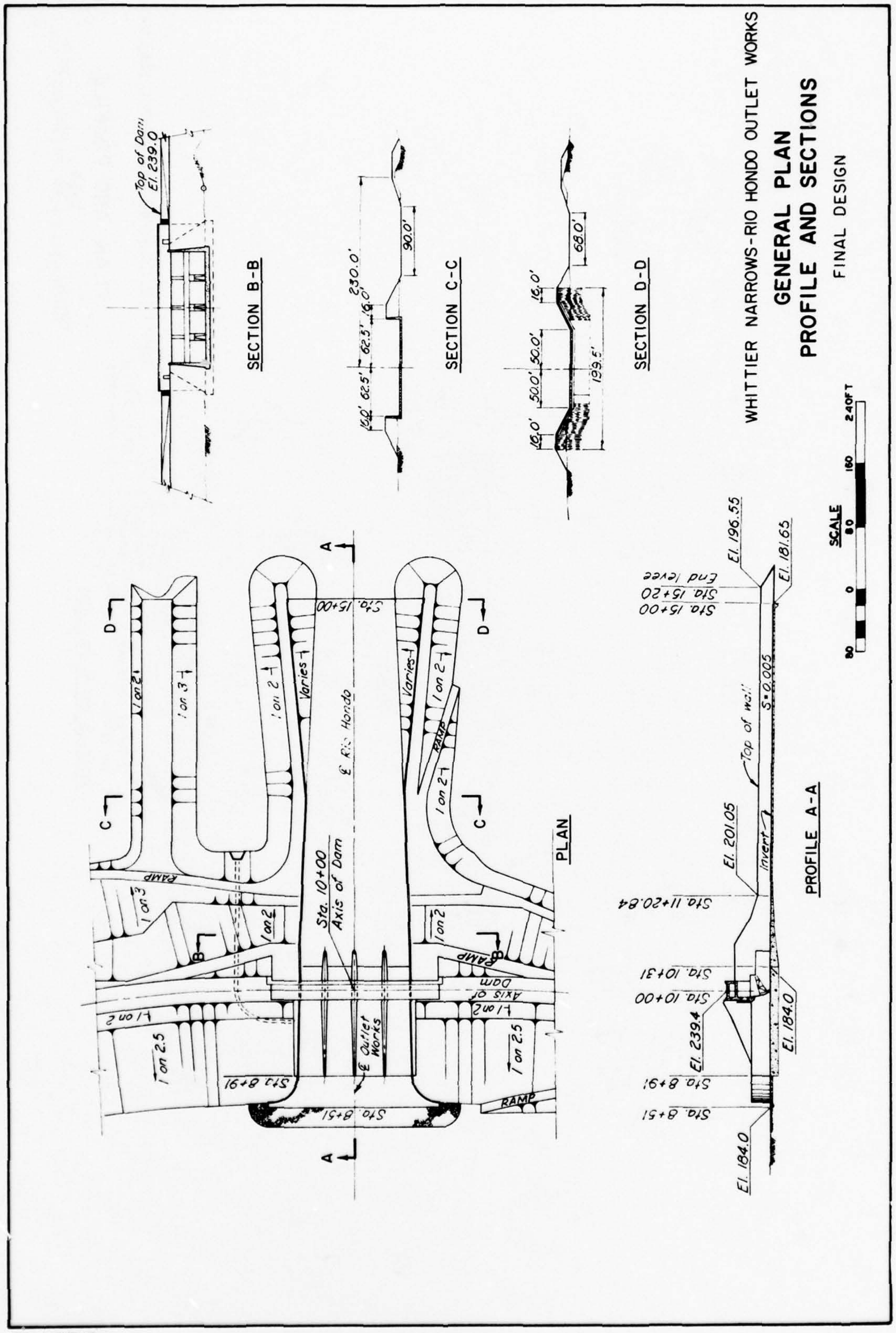
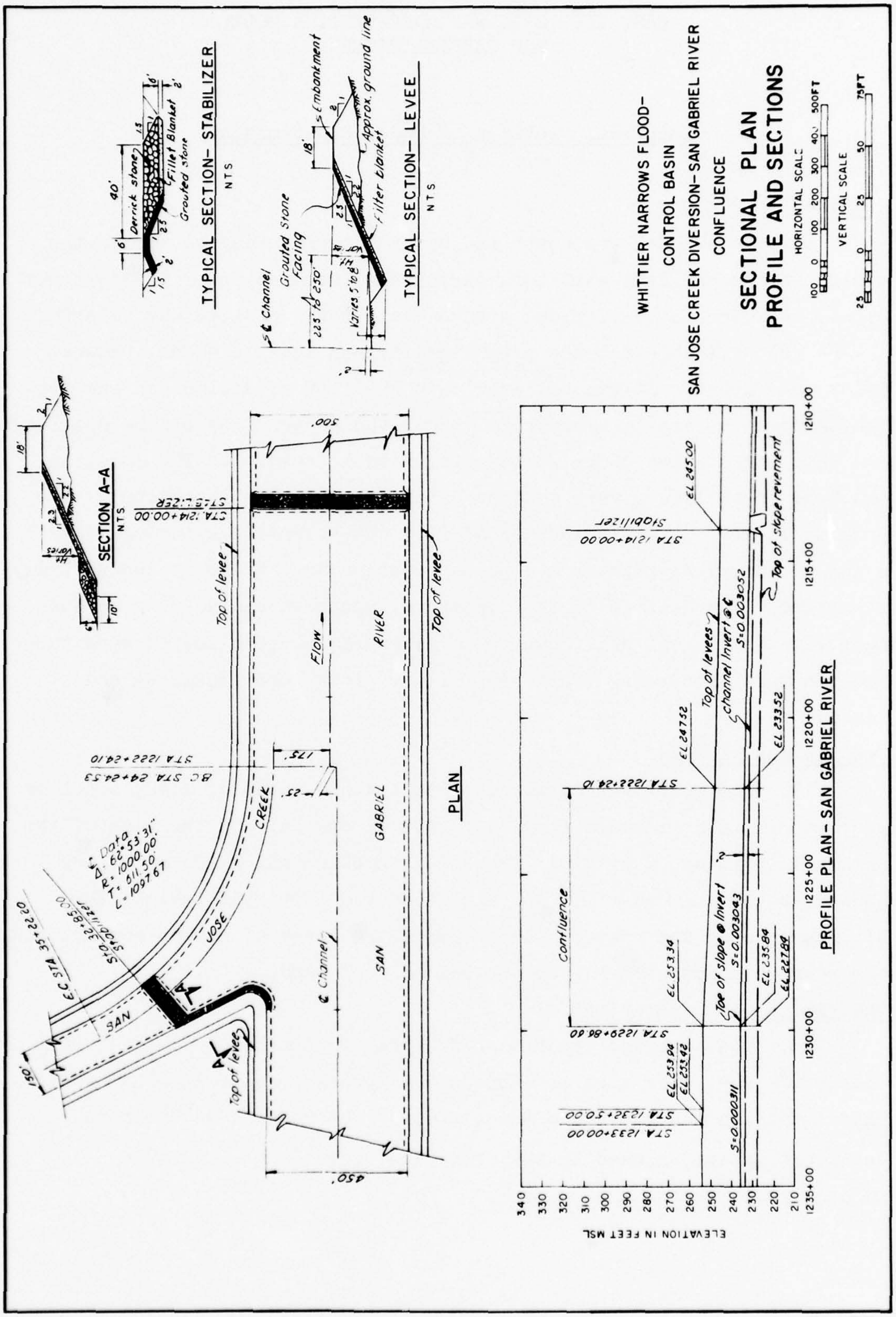

WHITTIER NARROWS-RIO HONDO OUTLET WORKS
GENERAL PLAN
PROFILE AND SECTIONS
FINAL DESIGN
SECTION B-B
Top of Dam
El. 239.0
SECTION C-C
230.0'
90.0'
16.0'
62.5'
62.5'
16.0'
SECTION D-D
16.0'
68.0'
50.0'
50.0'
16.0'
199.5'
PLAN
Sta. 10+00
Axis of Dam
℄ Rio Hondo
℄ Outlet Works
Sta. 8+91
Sta. 8+51
Sta. 15+00
1 on 2
1 on 3
1 on 2.5
Varies
RAMP
PROFILE A-A
El. 196.55
El. 181.65
Sta. 15+20
End levee
Sta. 15+00
S=0.005
Top of wall
Invert
El. 201.05
Sta. 11+20.84
Sta. 10+31
Sta. 10+00
El. 239.4
El. 184.0
Sta. 8+91
Sta. 8+51
El. 184.0
SCALE
80
0
80
160
240FT

PLATE 5


WHITTIER NARROWS FLOOD-
CONTROL BASIN
SAN JOSE CREEK DIVERSION–SAN GABRIEL RIVER
CONFLUENCE
SECTIONAL PLAN
PROFILE AND SECTIONS
HORIZONTAL SCALE
VERTICAL SCALE
TYPICAL SECTION– STABILIZER
NTS
Derrick stone
Fillet Blanket
Grouted stone
TYPICAL SECTION– LEVEE
NTS
Grouted Stone Facing
Embankment
Approx. ground line
Filter blanket
SECTION A-A
NTS
PLAN
Top of levee
Top of levees
Flow
SAN
GABRIEL
RIVER
SAN
JOSE
CREEK
℄ Channel
STA 1214+00.00
STABILIZER
STA 1222+24.10
BC STA 24+24.53
E.C STA 35+22.20
STA 32+85.00
Stabilizer
℄ Data
Δ: 62°53'31"
R= 1000.00
T= 611.50
L= 1097.67
PROFILE PLAN– SAN GABRIEL RIVER
ELEVATION IN FEET MSL
Confluence
Stabilizer
Top of levees
Channel Invert @ ℄
S=0.003052
Top of slope revetment
Toe of slope @ Invert
S=0.003043
S=0.000311
STA 1233+00.00
STA 1232+50.00
STA 1229+86.60
STA 1222+24.10
STA 1214+00.00
EL 245.00
EL 247.52
EL 233.52
EL 253.34
EL 235.84
EL 227.84
EL 253.94
EL 253.42
1235+00
1230+00
1225+00
1220+00
1215+00
1210+00

PART II: SPILLWAY HALF-SECTION MODEL SAN GABRIEL RIVER

Technical Features of Model Construction

Required conditions

11. This model was conceived as an interim study to verify the design of the spillway structure during the preparation of the contract plans. A model (or half-plan) with a scale ratio of 1:42 was selected as the most suitable for the pump capacity and available fall between inlet and outlet. It was believed that the flow conditions in the prototype would be nearly symmetrical about the center line of the spillway and that a half-plan model would give suitable results. The location and type of structure were design features previously determined. The problem involved a determination of flow characteristics through the spillway energy dissipator and probable scour downstream of the spillway structure. The details of the structure, upstream and downstream quadrant wing walls, end sill, cutoff wall, and downstream levees were subject to model analyses. Various spillway plans were simulated and tested.

Description of model

12. The model was constructed to a scale ratio of 1:42, model to prototype. It reproduced four half bays or one half of the plan of the proposed spillway, a portion of the east embankment, 1,200 ft of the channel below the end sill, and 510 ft of the left side levee. The spillway proper was constructed of wood, the levee of cement mortar, and the streambed downstream from the end sill of sand.

Theoretical considerations

13. The accepted equations of hydraulic similitude, based upon the Froude law, were used to express the mathematical relations between the dimensions and hydraulic quantities of the model and the prototype. The scale relations used in the study are shown in the following tabulation:

Dimension	Ratio	Scale Relation
Length	L_r	1:42
Area	$A_r = L_r^2$	1:1,764
Velocity	$V_r = L_r^{1/2}$	1:6.48
Time	$T_r = L_r^{1/2}$	1:6.48
Discharge	$Q_r = L_r^{5/2}$	1:11,432

Model appurtenances

14. Water used in the model operation was supplied from a recirculating system. A venturi meter installed in the inflow line was used to measure the required flow that was conveyed into the forebay and thence to the model area. A tailgate at the downstream end of the model was used to facilitate the flooding and draining of the model. Reservoir pool elevations, spillway water depths, and scour depths in the downstream channel were determined with point gages. Velocities were measured with a pitot tube.

15. Sand of uniform grain size was placed below the end sill and molded to the profile of the prototype channel to provide a rapid and comparative indication of erosion resulting from the various spillway designs. No attempt was made to simulate the bed material of the prototype. General scour results are considered to be qualitative. As the bed material of the prototype and model was considered to be easily erodible, the depth of localized scour was assumed to be quantitative.

Procedure and Accomplishment of Spillway Model Tests Half-Section

Procedure of tests

16. Six alternative spillway plans were tested in the model. Features tested for hydraulic verification included straight and quadrant wing walls in the inlet, right angle and quadrant wing walls in the outlet, and outlet channel with and without dentates. In all plans, the end sill was at el 194.0 and the design of the piers was unchanged. General details of the alternative plans are shown in Plate 7.

17. Tests of the spillway plans A-F were made to determine the effect on scouring action in the channel below the end sill under free-flow conditions. This procedure was considered reasonable since more severe conditions than were contemplated might be found to exist in the prototype due to the highly erodible quality of the streambed material. No rock protection was used below the end sill. No attempt was made to obtain quantitative data during the tests; therefore model tests were accomplished by visual observation and results were recorded by photographs. Spillway plan B was tested, but no data or photographs were taken.

18. The streambed downstream from the end sill was molded in sand to provide a rapid means of comparing the scouring tendencies of the various spillway plans. To obtain comparable scours of the sand bed for each spillway plan tested, all tests were of 20-min duration simulating a 2-hr operation (prototype). The prototype discharge simulated during the tests was 254,000 cfs. Table 1 gives all tests in order of the testing procedure with identifying descriptions, results, and illustrative references. The alternative plans are summarized below.

19. Spillway plan A. This plan, Photo 1, was a plain-type spillway with straight wing walls upstream, an offset levee downstream, and no dentates at the end sill. Photo 2 is a visual record of the discharge as it passed through the piers and of the flow conditions in the channel downstream and of the resulting scour. Test of this plan with prototype discharge of 254,000 cfs indicated that the spillway was not effective since the turbulence below the end sill produced excessive scour, the sand being scoured away to form a hole of considerable depth along the entire width of the end sill.

20. Spillway plan C. In this plan, the straight wing walls used in plan A were revised to quadrant walls with a radius of 35 ft. A row of 5-ft-high, 5-ft-wide, by 15-ft-long dentates, spaced on 10-ft centers, was added to the end sill. Photo 3 shows that flow conditions through the spillway structure were improved; however, the smoother but slower drawdown resulted in higher water surface to concentrate at the left bay of the spillway which would have a detrimental effect on the velocity

distribution downstream. Maximum scour occurred on the right side of the model (Photo 3).

21. Spillway plan D. This plan was the same as plan C, except that the dentates were increased from 5 ft to 8 ft in height. Increasing the height of dentates improved the hydraulic action by decreasing the height of jump and by moving the jump downstream and caused a great decrease in maximum depth of scour. The hydraulic action held the material up against the downstream face of the cutoff wall at the end sill. The scour was gradually sloped from the end sill with the deepest point well removed from the end of the structure. However, scour at the left side of the structure did occur because of the eddy action between the spillway wall and offset levee which had a tendency to carry away sand and expose the cutoff wall at that location. In general, this plan gave favorable results insofar as providing adequate protection for the spillway structure. For flow conditions and scour results, see Photo 4.

22. Spillway plan E. In this plan, the alignment of the levee was revised. The levee, which was previously offset from the spillway structure, was joined to the spillway wall. At the same time, the levee face was warped, with the upstream face vertical and the downstream face sloped 1V on 2.25H. In the test, the hydraulic action below the end sill was considerably improved. Although this plan did show much improvement hydraulically over those previously tested, excessive scour caused the sand to be moved away from along the toe of the levee. The general results were similar to those obtained in plan C. This plan together with views of its hydraulic performance and scour are shown in Photo 5.

23. Spillway plan F. This plan was the same as plan D except that the downstream end of the spillway wall was terminated by a quadrant wall with a 75-ft radius. In general, this dentated plan was found to produce the most favorable hydraulic action and shallower scour than those previously tested. This plan, which gave the best results, was considered satisfactory. The quadrant wall at the downstream end of the spillway wall reduced much of the eddy action observed in plans A, C, and D and, consequently, the deep scour along the toe of the levee.

General view of the model, hydraulic performance, and resulting scour are shown in Photo 6. The scour pattern for this plan is shown in Plate 8. Water depth contours for 254,000 cfs are shown in Plate 9; water-surface cross sections are shown in Plate 10.

24. Additional tests on spillway plan F. Tests were conducted to determine the depth of cutoff wall at the downstream end of the spillway and the extent of rock protection required downstream of the end sill and along the toe of the levee. Six tests were made on three types of rock apron designs, one being a 10-ft-thick, plain rock apron on a 1V-on-2H slope, extending to a depth of 20 ft; the other two, designated as rock protection plan 1 and rock protection plan 2, are shown in Plate 7. Spillway plan F (recommended) was used in the testing. Testing of each type apron design was made with a prototype discharge of 254,000 cfs and a 30-min (model) duration. Also, tests simulating discharges of 150,000 and 200,000 cfs as well as a design-flood hydrograph were made of rock protection plan 2 (recommended plan). Results of these tests are shown in photographs and scour patterns. Table 2 gives a resumé of the results of each rock apron test with illustrative references. Rock protection tests are summarized below.

25. Test with plain rock apron (254,000 cfs). Scour pattern was obtained for a discharge of 254,000 cfs. A 30-min operation at a discharge of 254,000 cfs resulted in displacement of rock only in the vicinity of the quadrant wall. The maximum depth of scour was 40 ft at a point 140 ft downstream from the end sill and 300 ft from the toe of left levee. Flow conditions together with a view of the scour after the 30-min run are shown in Photo 7. Scour pattern is shown in Plate 11.

26. Test with rock protection plan 1 (254,000 cfs). Since the previous test had indicated that the rock would be displaced in the vicinity of the quadrant wall, this area was heavily armored with graded derrick stone as shown in Photo 8. Comparison of the scour results of this test with those produced by the previous test showed that with 254,000 cfs the general pattern of scour was improved since it was much shallower. The maximum depth of scour was 34 ft, at a point 110 ft downstream from the end sill and 160 ft from the toe of left levee.

General views of the model are shown in Photo 8. Photographs of the flow and scour below spillway are shown in Photo 9; the scour pattern is shown in Plate 12.

27. Test with rock protection plan 2 (254,000 cfs). The rock protection was revised to conform to rock plan 2 (Plate 7). General views of the model together with rock placement are shown in Photo 10; scour photographs are shown in Photo 11. The results were similar to those with rock plan 1 but less severe in the area adjacent to the quadrant wall. Maximum depth of scour was 42 ft, 125 ft downstream from the end sill and 225 ft from the toe of left levee. Scour pattern is shown in Plate 13. This rock plan gave assurance of providing adequate protection for the stability of the spillway structure.

28. In addition to the foregoing test, tests simulating a design-flood hydrograph and prototype discharges of 150,000 and 200,000 cfs were made on rock plan 2. Radial gates were installed to regulate the flows through the spillway. The height of gate was determined by the requirement that in the closed position, the tops of the gates would be at reservoir design water-surface el 229.0. Results of these tests are briefly discussed in the following subparagraphs.

a. With a discharge of 200,000 cfs the overall flow conditions through the spillway section were satisfactory. Flow conditions through the spillway with pool el 231.5 are shown in Photo 12; for this discharge (200,000 cfs) the gates were opened 20.7 ft. The scour results after a 1-hr run (model time) were generally similar to those obtained with 254,000 cfs and a 30-min run. However, the deepest scour hole in this test was slightly less. The maximum depth of scour was 40 ft, 125 ft downstream from the end sill and 280 ft from the toe of left levee. Scour photographs are also shown in Photo 12; scour pattern is shown in Plate 14.

b. With a discharge of 150,000 cfs (Photo 13) and a 1-hr 50-min (model) operation, the depth of maximum scour was 34 ft. It occurred 110 ft downstream from the end sill and 170 ft from the toe of the left levee. Results of the scour are shown in Photo 13 and Plate 15.

c. A design-flood hydrograph was simulated in this test. Table 3 gives the range of discharges, corresponding pool elevations, and gate openings. The time of hydrograph run was 6 hr 41 min (model time). The hydrograph run is

shown in Photo 14; results of scour are shown in Photo 15. Plate 16 shows scour pattern. This test gave somewhat better results in the scour pattern than that produced by the previous tests. The scour was such as to be considered moderate but was satisfactory for stability of the spillway structure. Maximum scour of 40 ft occurred 155 ft from the end sill and 260 ft from the toe of left levee.

Conclusions

29. In general, results of the tests indicated that the scour patterns downstream from the end sill were essentially the same for all rock apron designs. However, severe scour at the left side adjacent to levee did not occur with rock plan 1 or rock plan 2 because of the heavy rock protection in that area. Spillway plan F equipped with dentates, quadrant wall downstream, and rock protection plan 2 gave the most favorable results.

Table 1

Summary of Test Data

Discharge 254,000 cfs

Model Tests 20-min Duration

Spillway Plan	Reference Photograph	Remarks
A	1 and 2	Maximum scour occurred on right side of model
B		Visual observation only
C	3	Stable flow conditions through piers. Excessive scour adjacent to levee
D	4	Material held against end sill. Moderate scour
E	5	Maximum scour occurred along and below warped levee. Structure endangered
F	6	Satisfactory flow conditions with quadrant wall. Stable flow. Minimum scour

Table 2

Resumé of Tests on Spillway Plan F

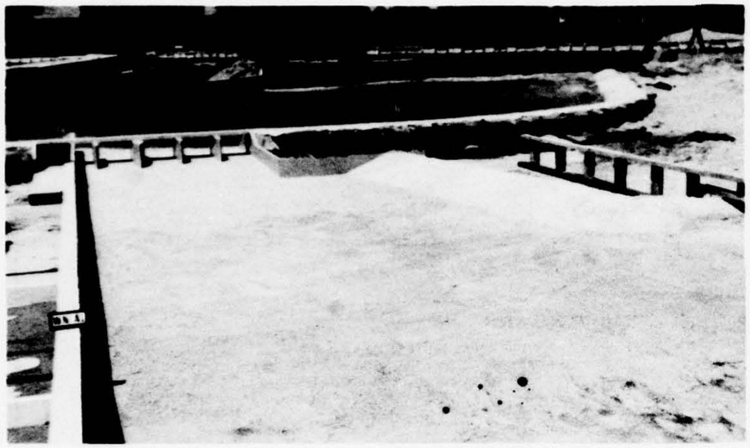
Type Rock Protection	Discharge cfs	Duration hr	Duration min	Scour Depth ft	Scour Location	References Photograph	References Plate
Plain rock apron	254,000	--	30	40	140 ft downstream from end sill and 300 ft from toe of left levee.	7	11
Rock plan 1	254,000	--	30	34	110 ft downstream from end sill and 160 ft from toe of left levee.	8 and 9	12
Rock plan 2	254,000	--	30	42	125 ft downstream from end sill and 225 ft from toe of left levee.	10 and 11	13
Rock plan 2	200,000	1	--	40	125 ft downstream from end sill and 280 ft from toe of left levee.	12	14
Rock plan 2	150,000	1	50	34	110 ft downstream from end sill and 170 ft from toe of left levee.	13	15
Rock plan 2	Hydrograph run 40,000-254,000	6	40	40	155 ft downstream from end sill and 260 ft from toe of left levee.	14 and 15	16

Table 3

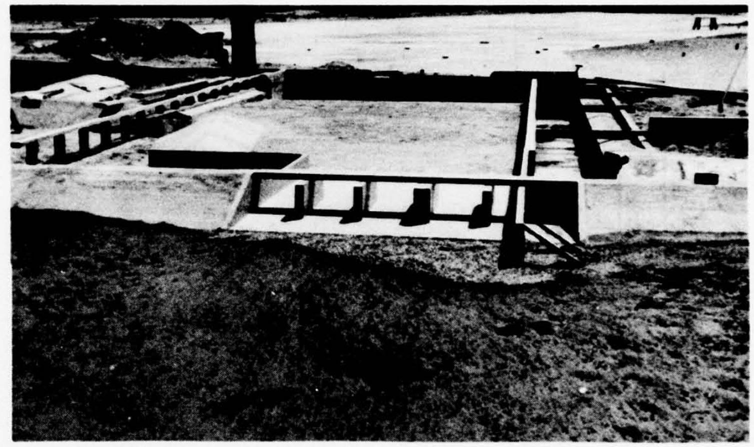
Spillway Design Flood Operation Schedule

Half-Section

Discharge, cfs	Pool Elevation ft msl	Gate Opening ft	Time Increments Proto hr	Proto min	Model hr	Model min
40,000	229.5	2.9	1	15	0	12
74,000	230.0	5.6	1	30	0	14
104,000	230.5	8.4	1	45	0	16
125,000	231.0	10.5	11	45	1	48
147,600	231.5	12.8	3	30	0	32
172,400	232.0	16.1	3	15	0	30
203,000	233.1	20.7	1	15	0	12
234,000	233.8	Fully open	0	30	0	5
244,000	234.0	Fully open	1	0	0	10
254,000	234.0	Fully open	1	30	0	14
244,000	234.0	Fully open	0	45	0	7
234,000	233.8	Fully open	0	45	0	7
203,000	233.1	20.7	1	0	0	10
172,400	232.0	16.1	0	45	0	7
147,600	231.5	12.8	0	30	0	5
125,000	231.0	10.5	1	0	0	10
104,000	230.5	8.5	1	30	0	14
74,000	230.0	5.6	4	30	0	42
40,000	229.5	2.9	5	0	0	45
					6	40

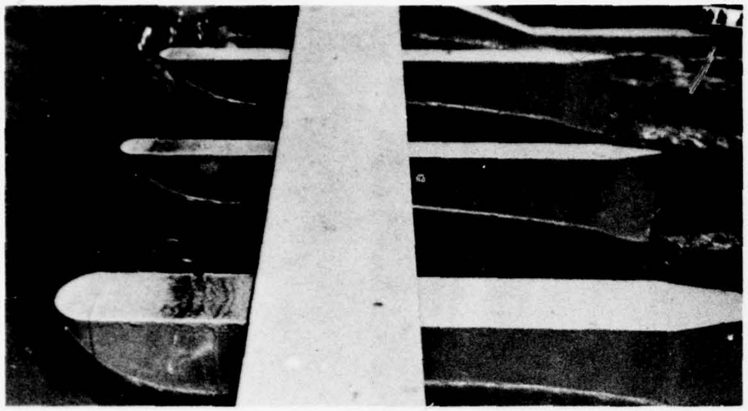


Levee offset, looking upstream

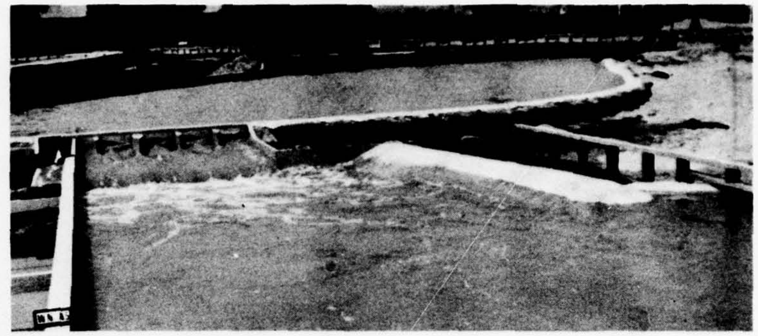


Looking downstream, note straight wing walls

Photo 1. Spillway plan A, straight wing walls upstream and no dentates

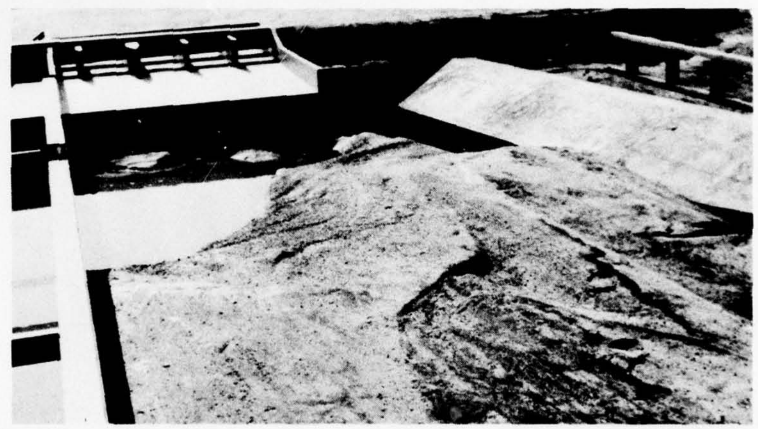


Drawdown at wing wall and piers



Looking upstream

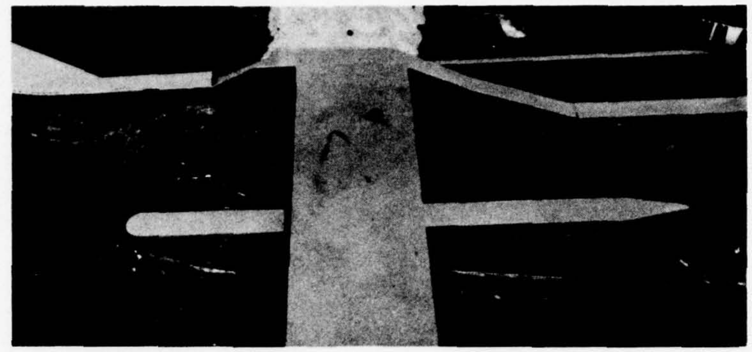
Scour below end sill, looking upstream



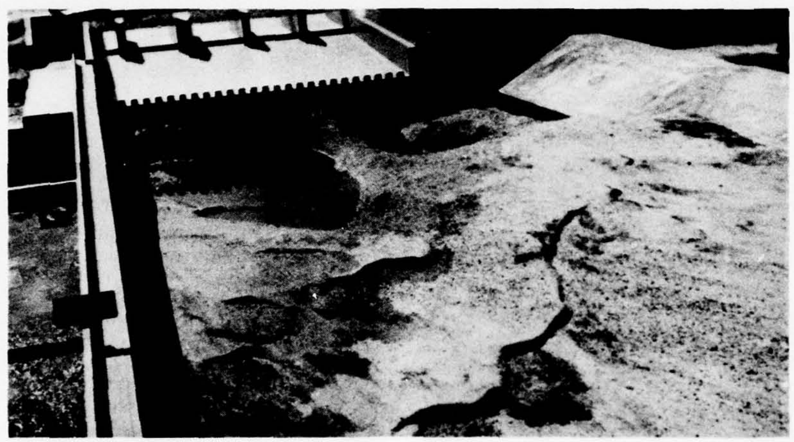
Scour below end sill, looking toward upstream right side

Photo 2. Spillway plan A, discharge 254,000 cfs; time of run 20 min

Peak discharge 254,000 cfs

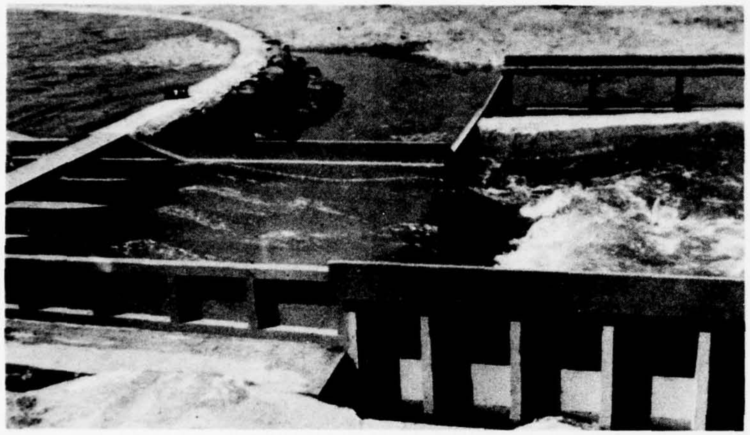


Drawdown at quadrant wall and pier, discharge 254,000 cfs

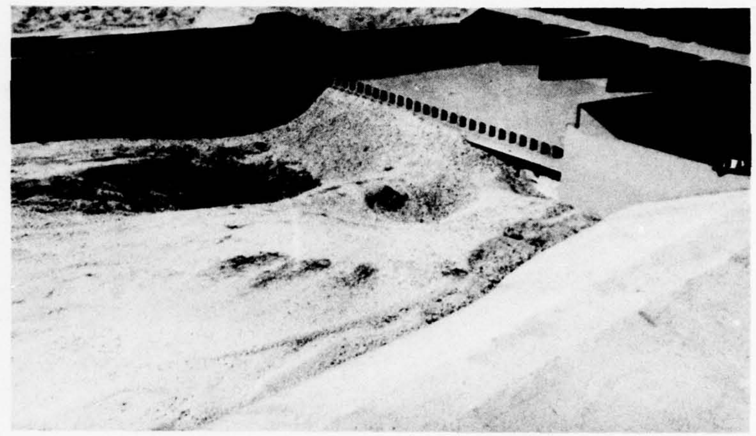


Scour after 20-min run

Photo 3. Spillway plan C, with quadrant walls upstream and 5-ft-high by 5-ft-wide dentates, spaced 5 ft apart

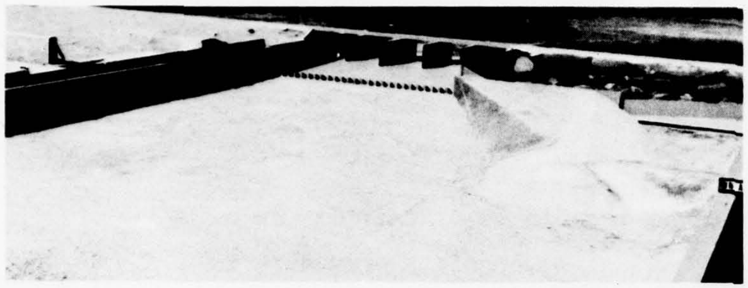


Peak discharge 254,000 cfs

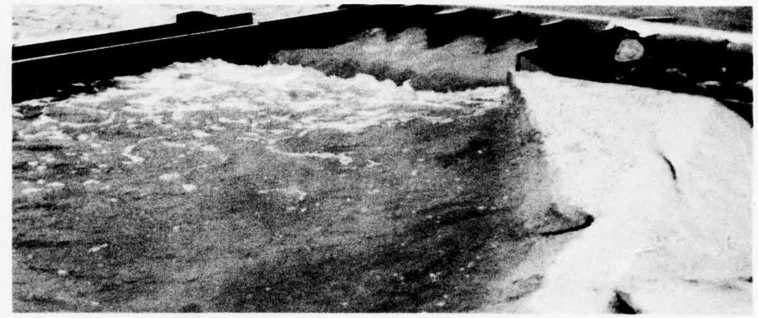


Scour after 20-min run

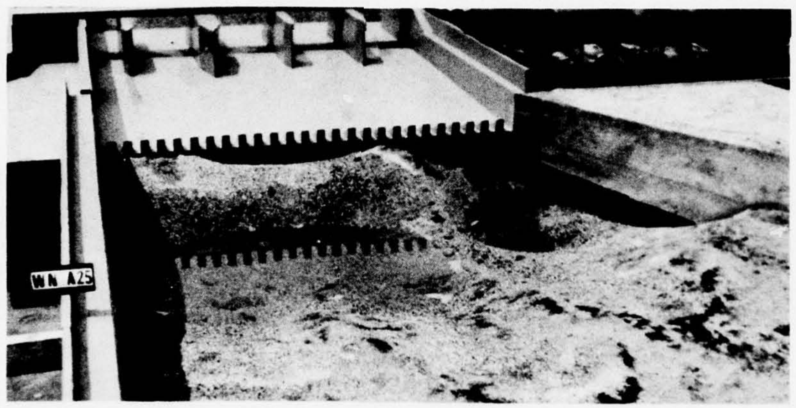
Photo 4. Spillway plan D, with quadrant walls upstream and 8-ft-high by 5-ft-wide dentates, spaced 5 ft apart



Looking upstream; note warped levee

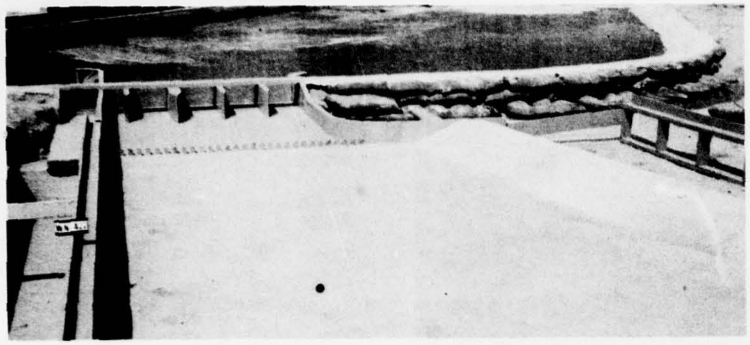


Peak discharge 254,000 cfs

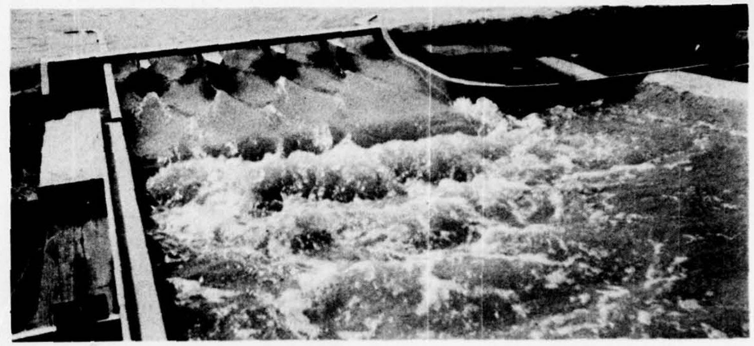


Scour after 20-min run

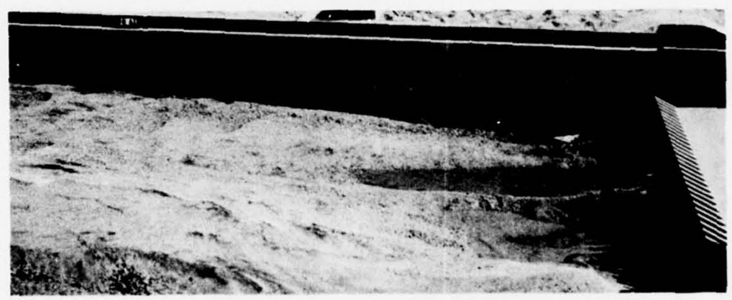
Photo 5. Spillway plan E, with quadrant walls upstream, 8-ft-high by 5-ft-wide dentates, spaced 5 ft apart, and warped levee downstream



Looking upstream

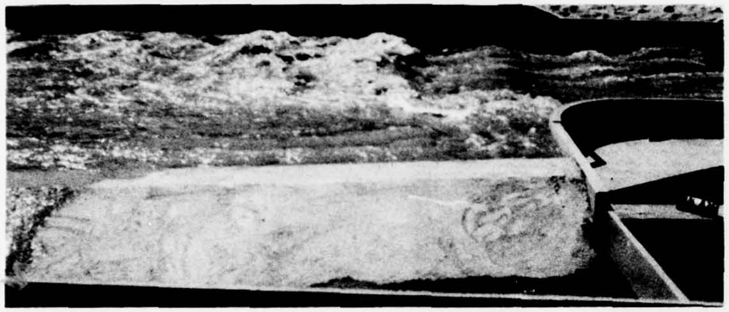


Peak discharge 254,000 cfs



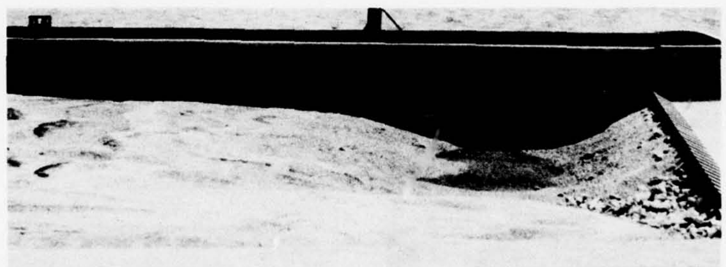
Scour after 20-min run

Photo 6. Spillway plan F, quadrant wall added downstream



Peak discharge 254,000 cfs

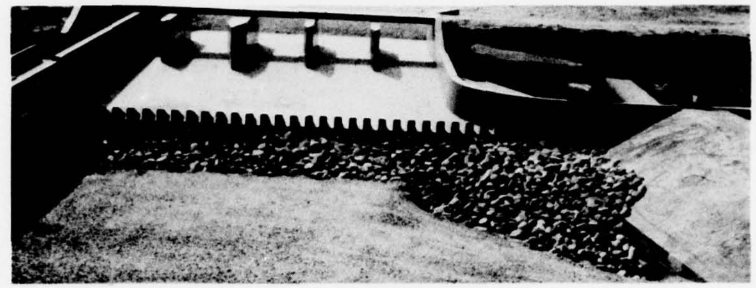
Downstream quadrant wall, discharge 254,000 cfs



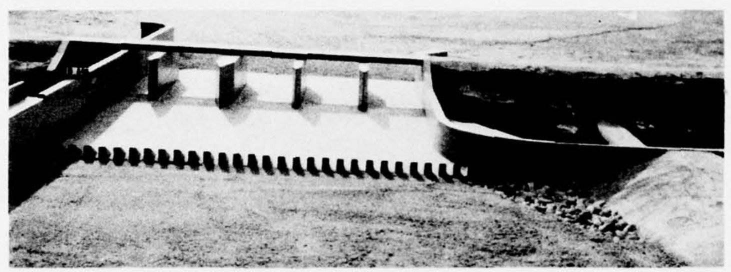
Scour after 30-min run

Photo 7. Spillway plan F, with 10-ft-thick blanket of rock protection extending to a depth of 20 ft below end sill

Looking at downstream quadrant wall and levee
before rock placement



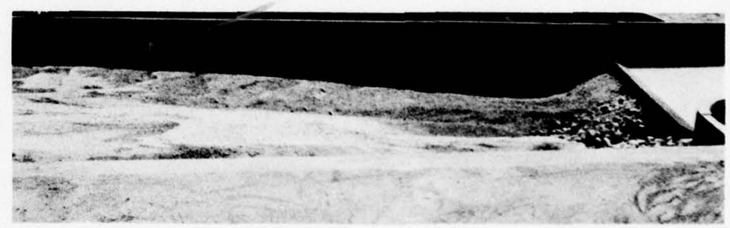
Looking upstream at rock placement



Looking upstream at sand backfill over rock protection

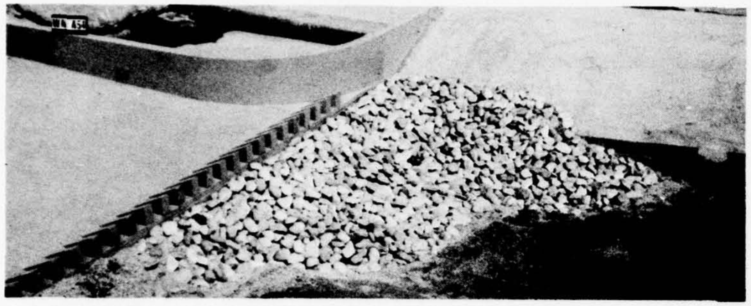
Photo 8. Spillway plan F, rock protection plan 1

Hydraulic action near quadrant wall,
discharge 254,000 cfs

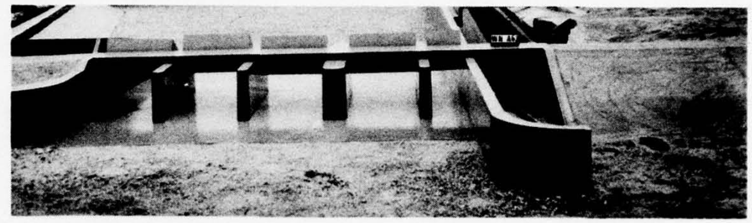


Scour after 30-min run

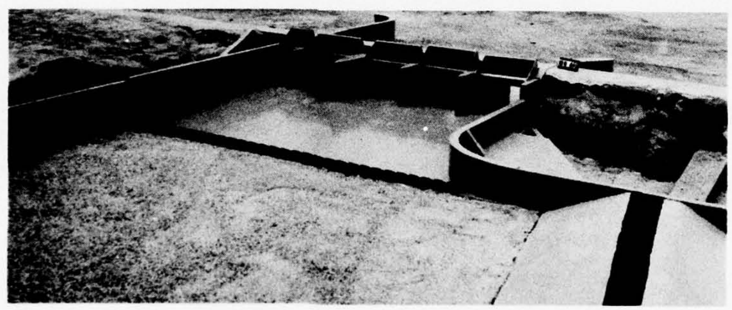
Photo 9. Spillway plan F, rock protection plan 1



Rock protection before backfill



Looking downstream; note quadrant wing walls



Looking upstream

Photo 10. Spillway plan F, rock protection plan 2

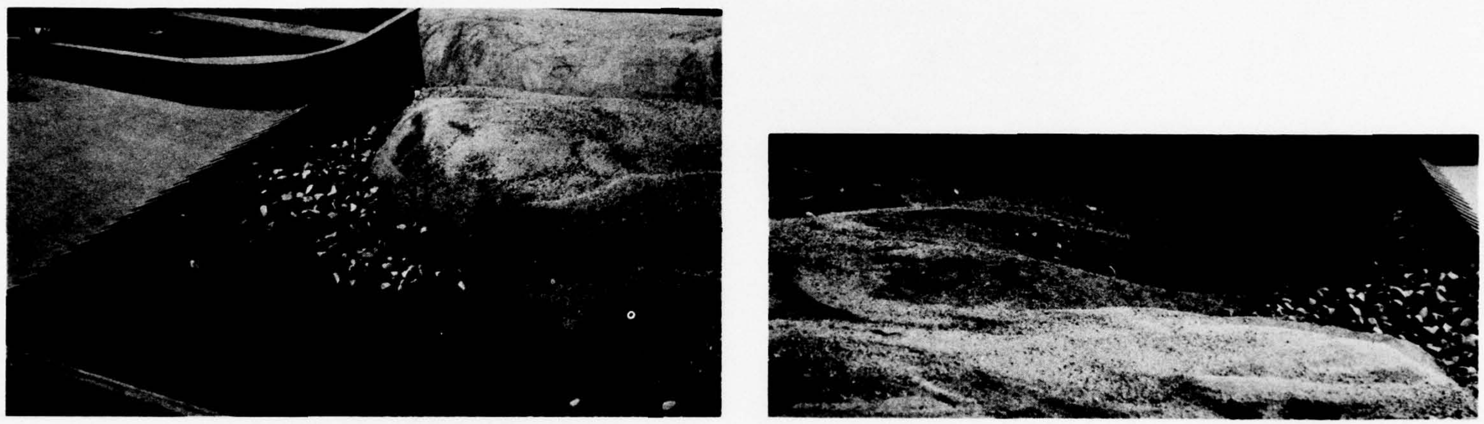
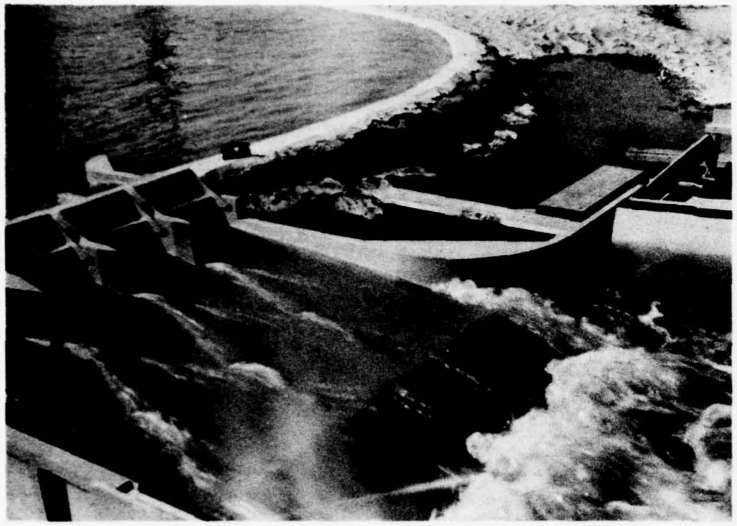
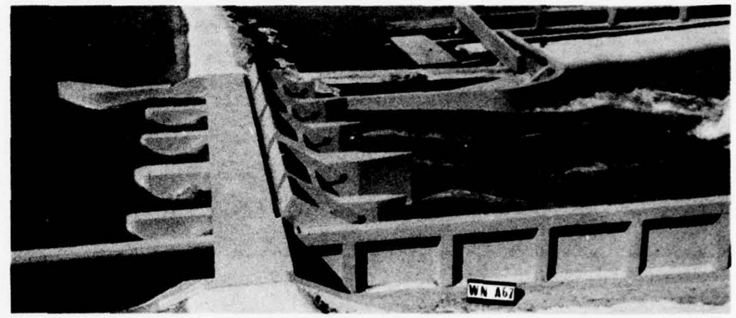
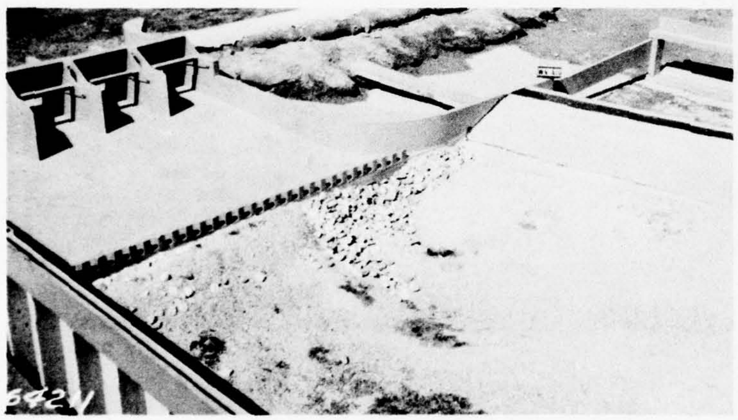


Photo 11. Spillway plan F, rock protection plan 2, scour after 30-min run, discharge 254,000 cfs



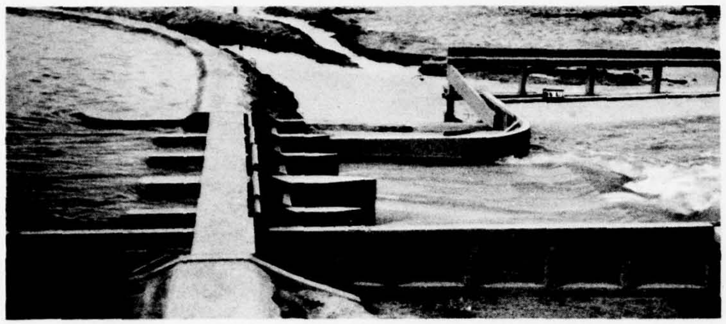


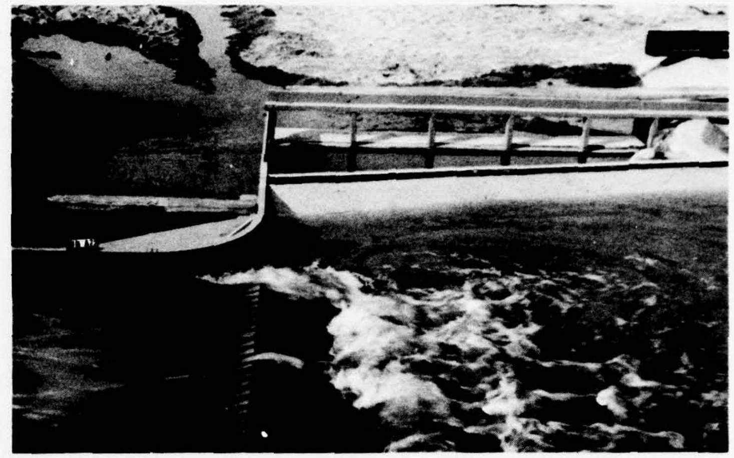
Pool el 231.5 and gates opened 20.7 ft



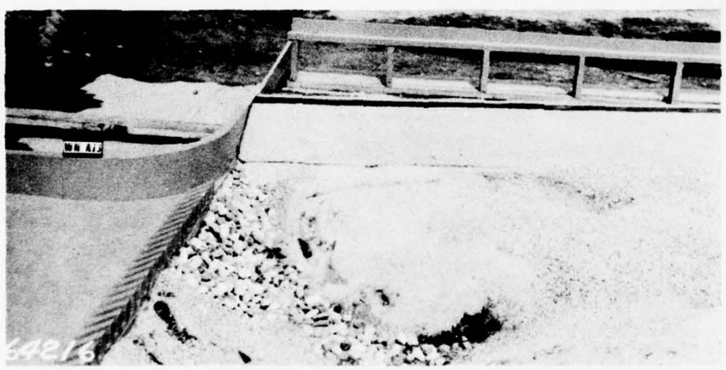
Scour after 1 hr

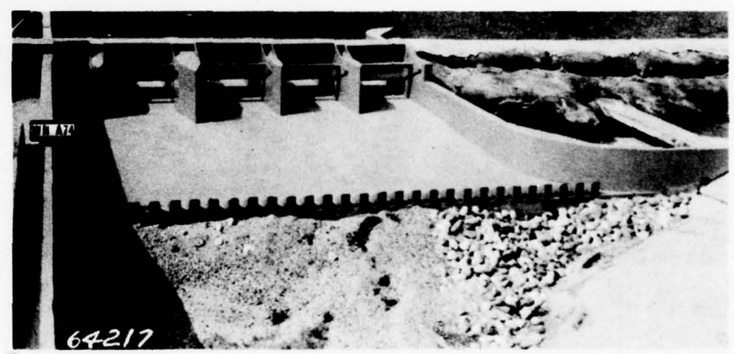
Photo 12. Spillway plan F, rock protection plan 2, discharge 200,000 cfs





Gates adjusted to hold pool elevation at 234.0

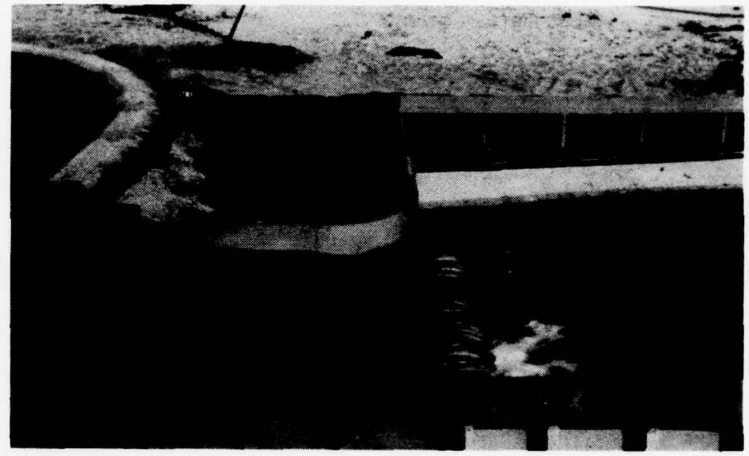




Scour after 1 hr 50 min

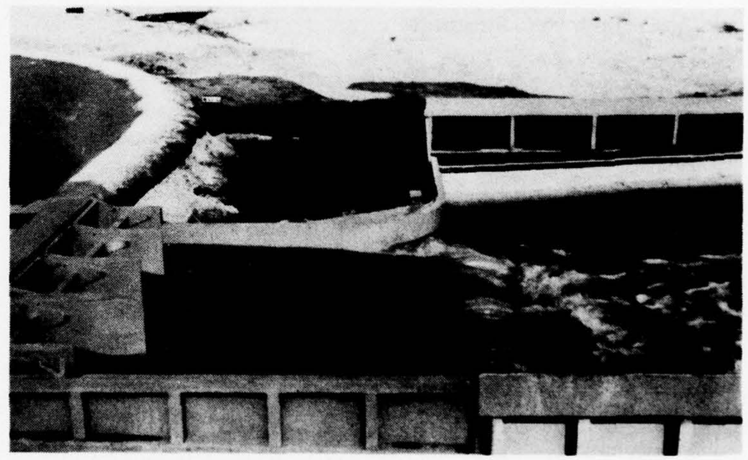
Photo 13. Spillway plan F, rock protection plan 2, discharge 150,000 cfs

40,000 cfs, gate opening 2.9 ft



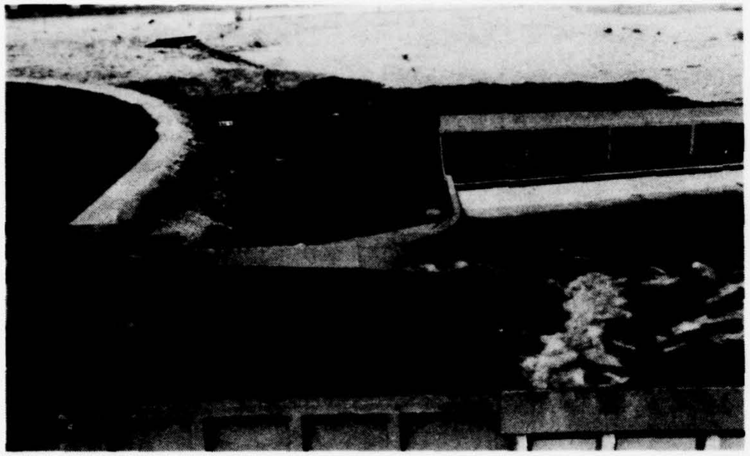
74,000 cfs, gate opening 5.6 ft

104,000 cfs, gate opening 8.4 ft

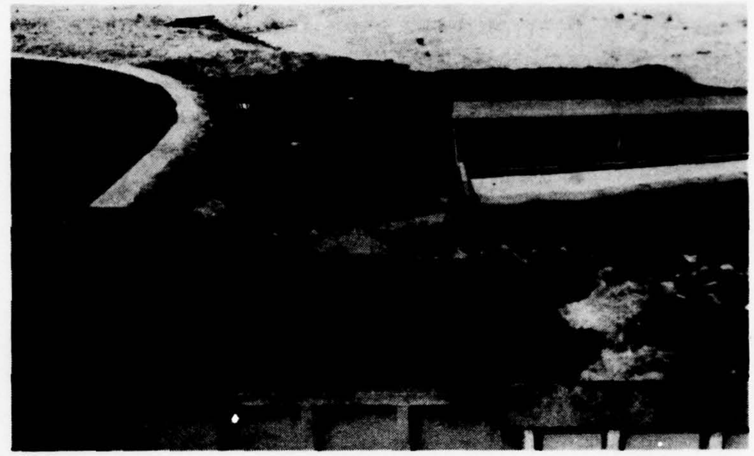


125,000 cfs, gate opening 10.5 ft

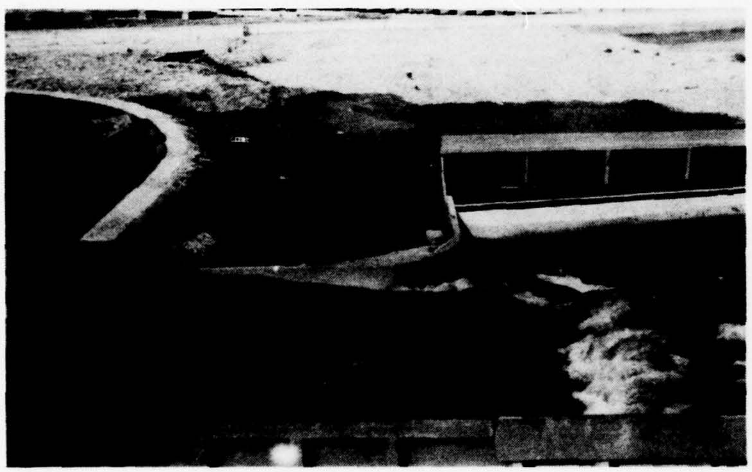
Photo 14. Spillway plan F, rock protection plan 2, hydrograph run (sheet 1 of 3)



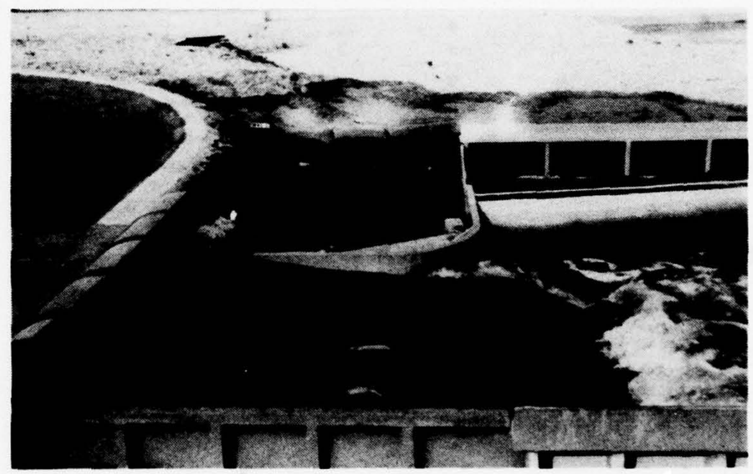
147,600 cfs, gate opening 12.8 ft



172,400 cfs, gate opening 16.1 ft

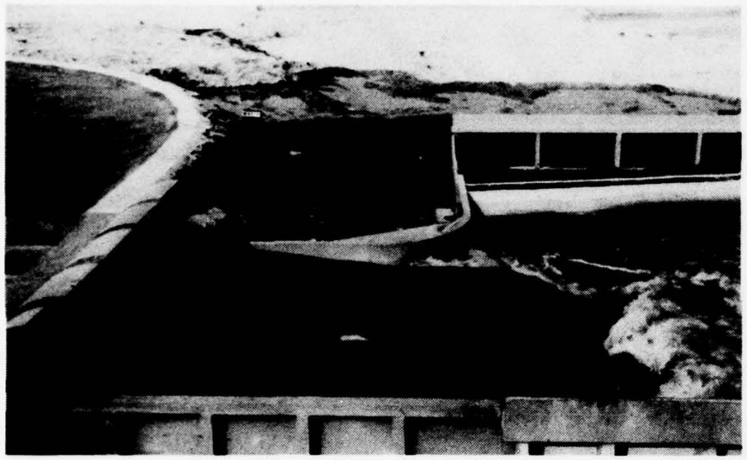


203,000 cfs, gate opening 20.7 ft

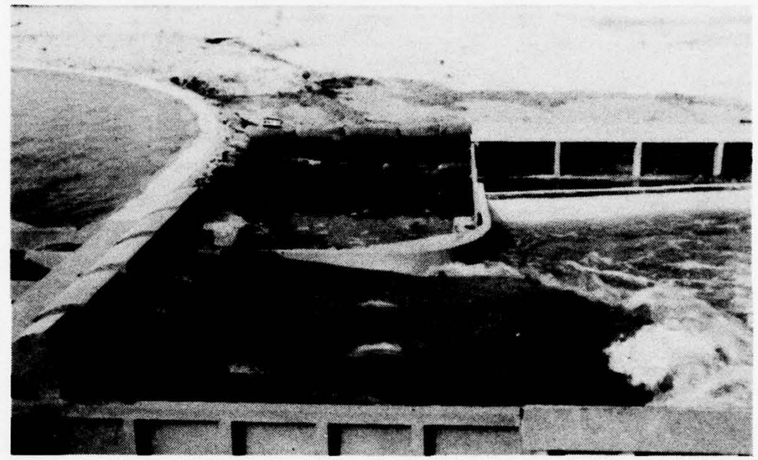


234,000 cfs, gates fully open

Photo 14 (sheet 2 of 3)



244,000 cfs

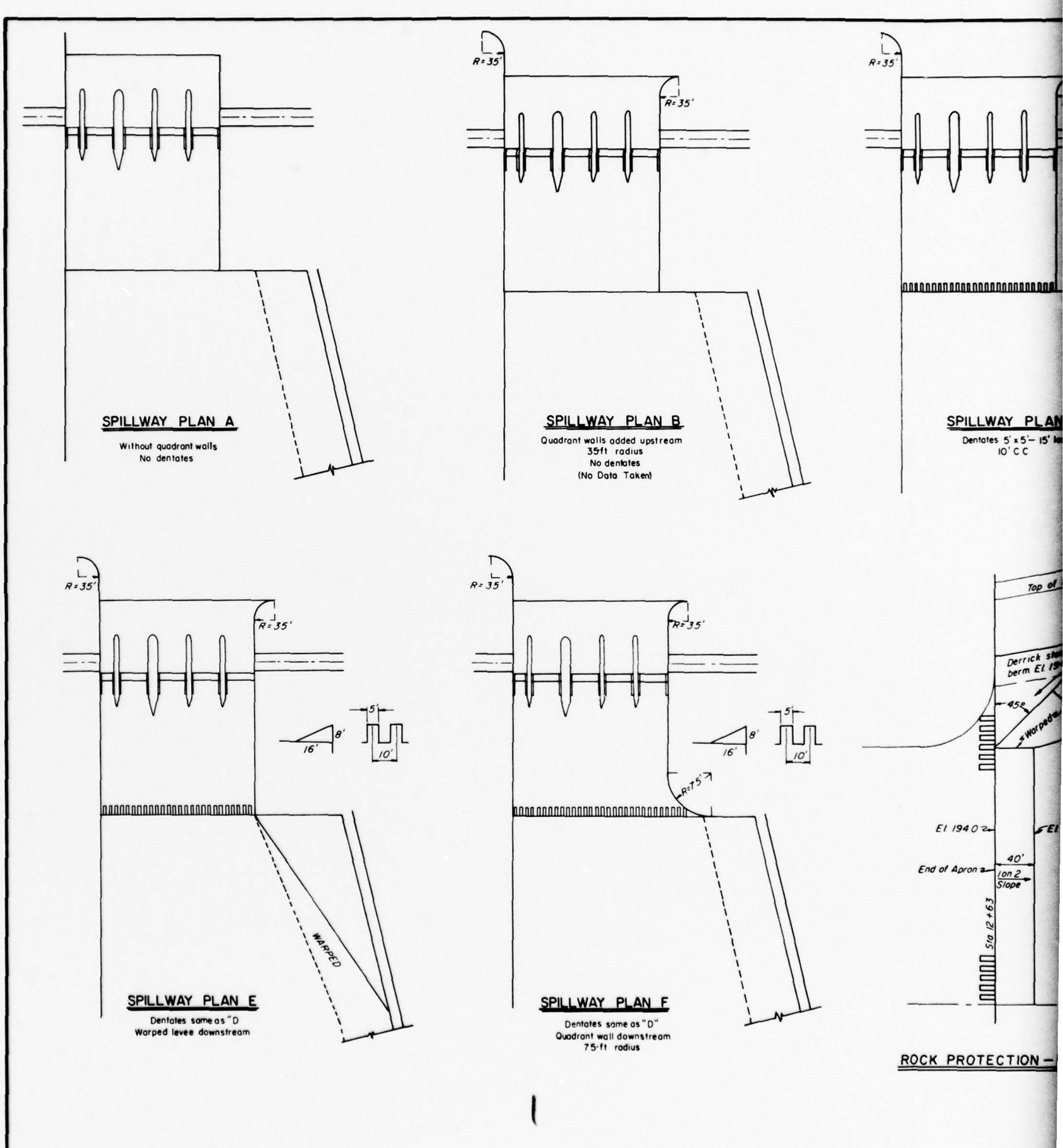


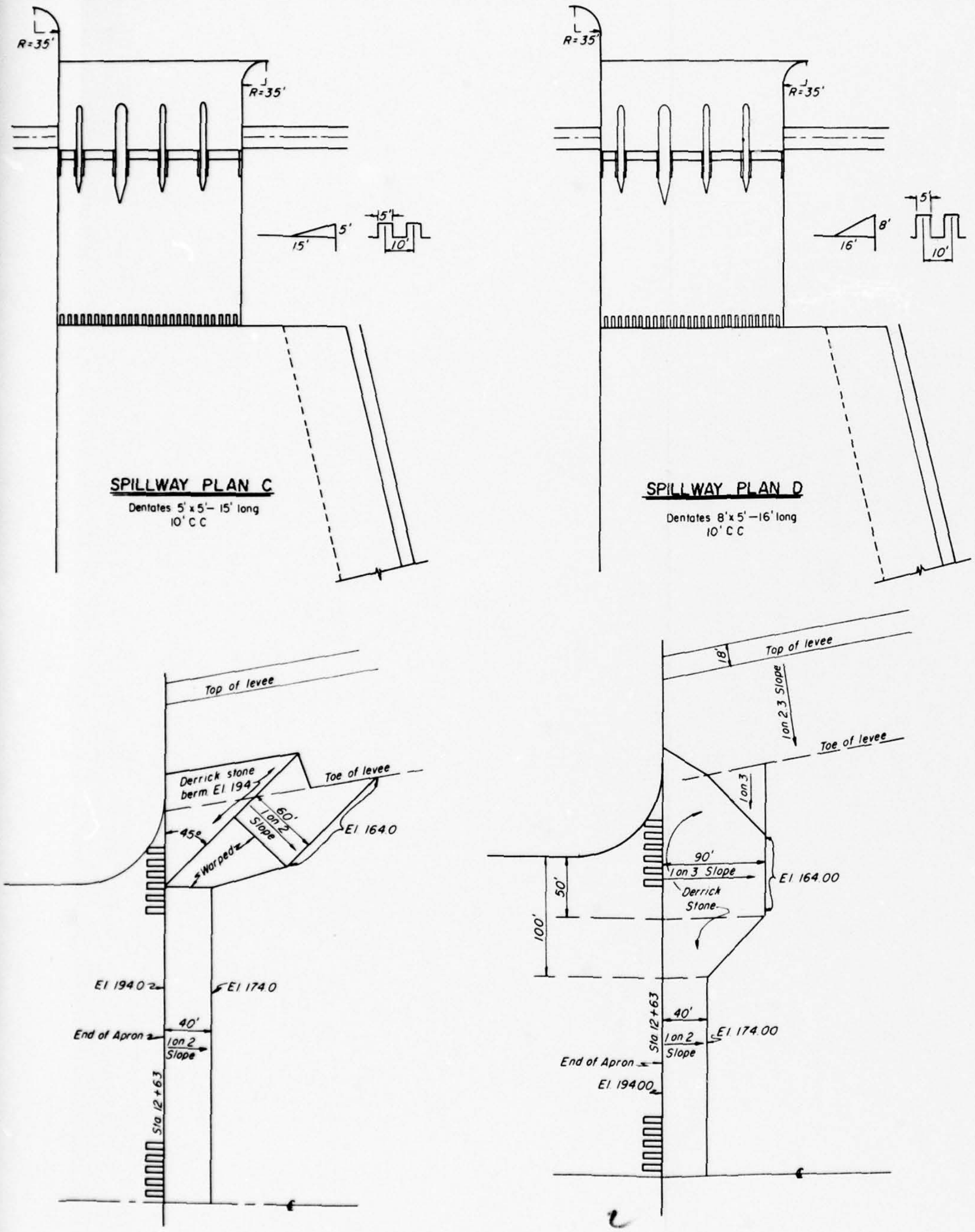
254,000 cfs

Gates fully open

Photo 14 (sheet 3 of 3)

Photo 15. Spillway plan F, rock protection plan 2, scour after hydrograph run; elapsed time 6 hr 40 min

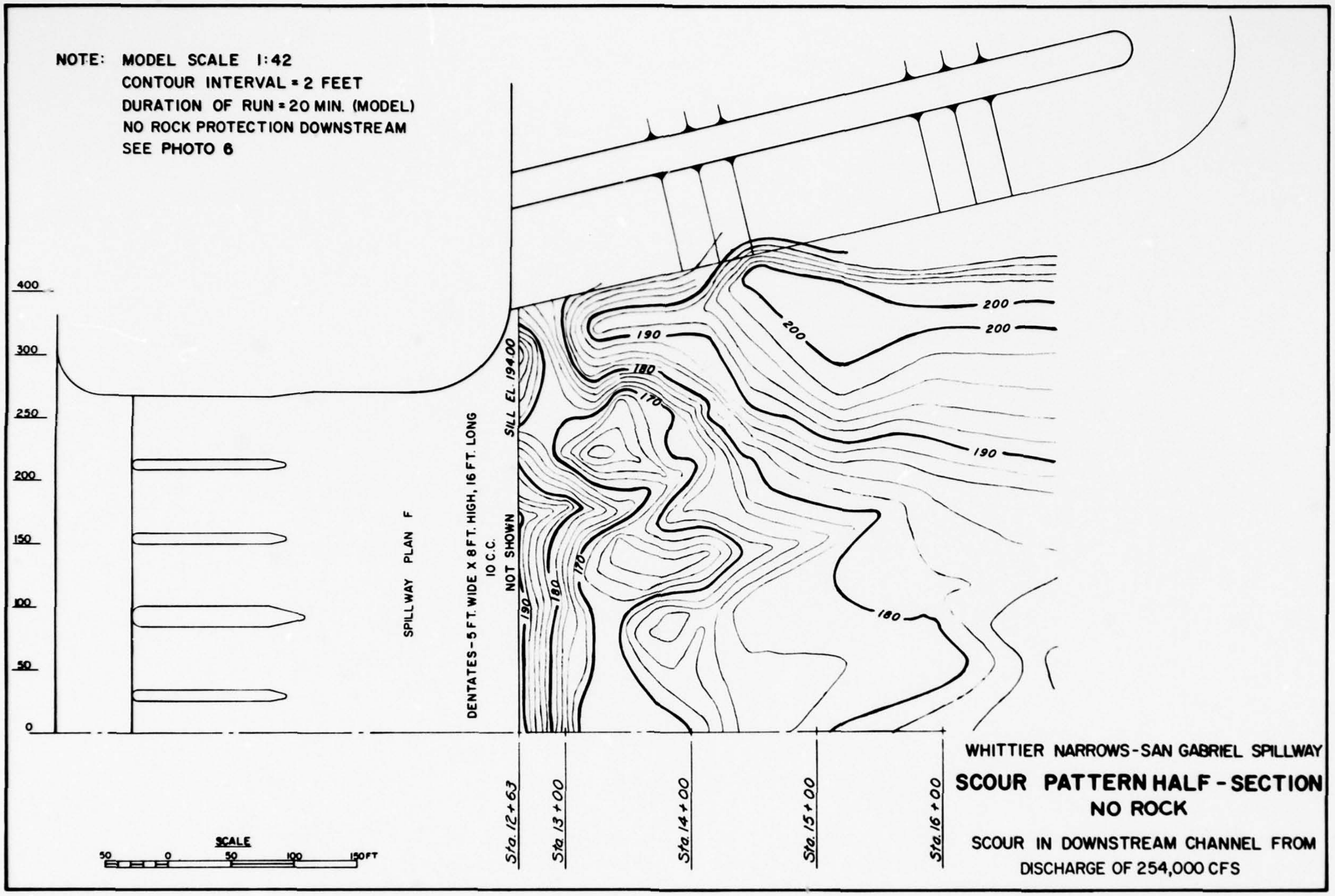

SPILLWAY PLAN A
Without quadrant walls
No dentates
R=35'
R=35'
SPILLWAY PLAN B
Quadrant walls added upstream
35ft radius
No dentates
(No Data Taken)
R=35'
SPILLWAY PLAN
10' C C
R=35'
R=35'
16'
8'
5'
10'
WARPED
SPILLWAY PLAN E
Dentates same as "D"
Warped levee downstream
R=35'
R=35'
16'
8'
5'
10'
R=75'
SPILLWAY PLAN F
Dentates same as "D"
Quadrant wall downstream
75-ft radius
45°
El 194.0
40'
End of Apron
1 on 2
Slope
Sta 12+63
ROCK PROTECTION –



SCALE

60 0 60 120 180 FT

WHITTIER NARROWS – SAN GABRIEL SPILLWAY
ALTERNATIVE PLANS A-F
HALF – SECTION
AND
ROCK PROTECTION PLANS 1 AND 2

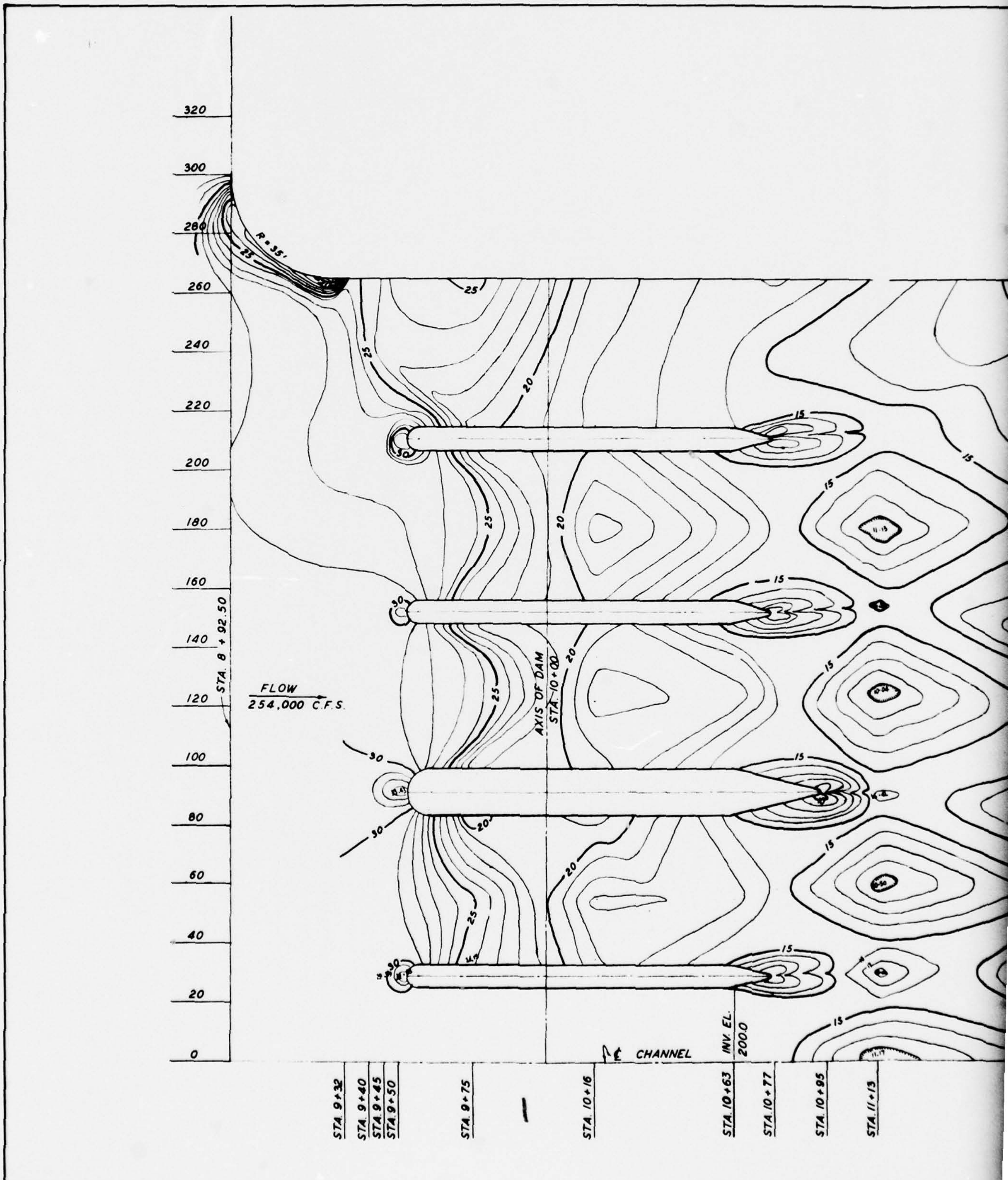


WHITTIER NARROWS-SAN GABRIEL SPILLWAY

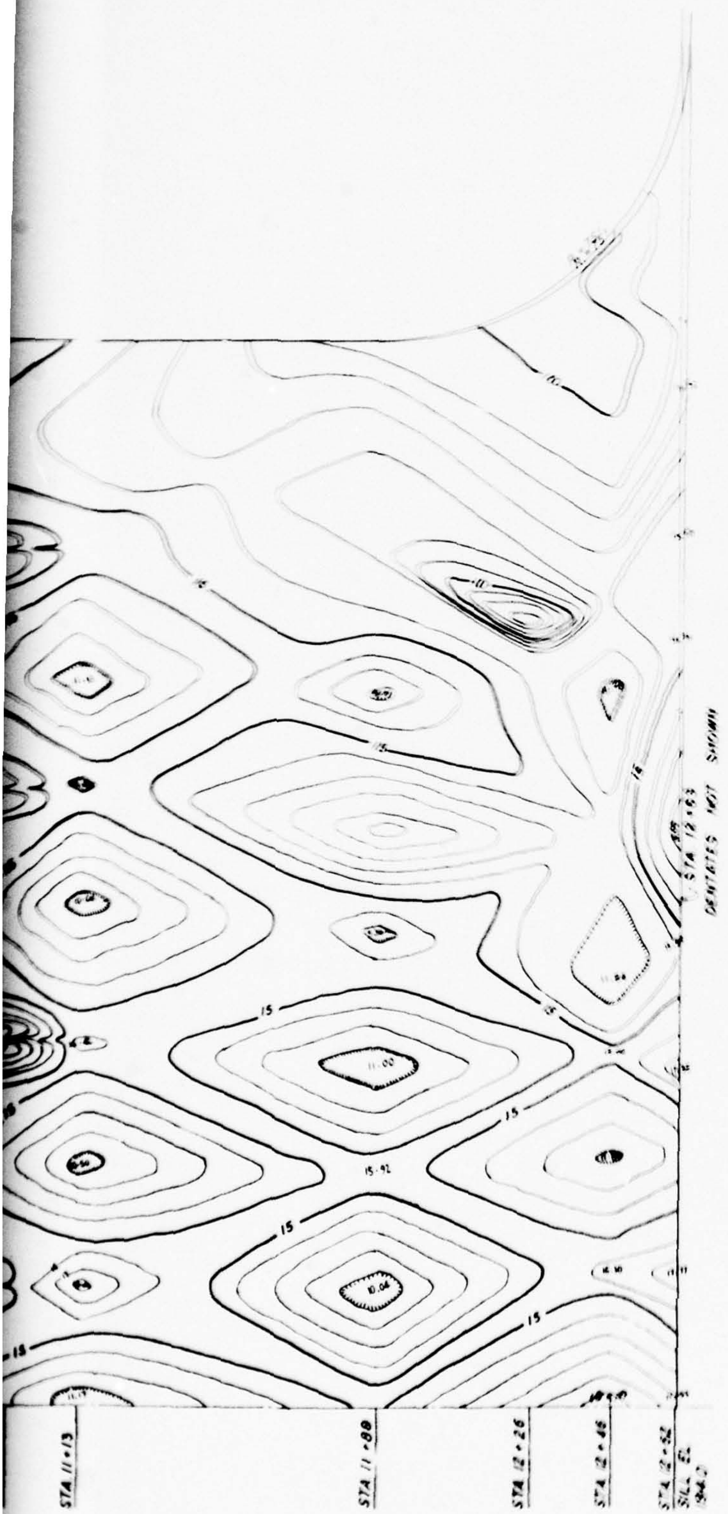
SCOUR PATTERN HALF-SECTION
NO ROCK

SCOUR IN DOWNSTREAM CHANNEL FROM
DISCHARGE OF 254,000 CFS

PLATE 8



320
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
R = 35'
STA. 8 + 92.50
FLOW
254,000 C.F.S.
AXIS OF DAM
STA. 10+00
℄ CHANNEL
INV. EL.
200.0
STA. 9+32
STA. 9+40
STA. 9+45
STA. 9+50
STA. 9+75
STA. 10+16
STA. 10+63
STA. 10+77
STA. 10+95
STA. 11+13



NOTE: SEE ALSO PLATE 10

SCALE

2

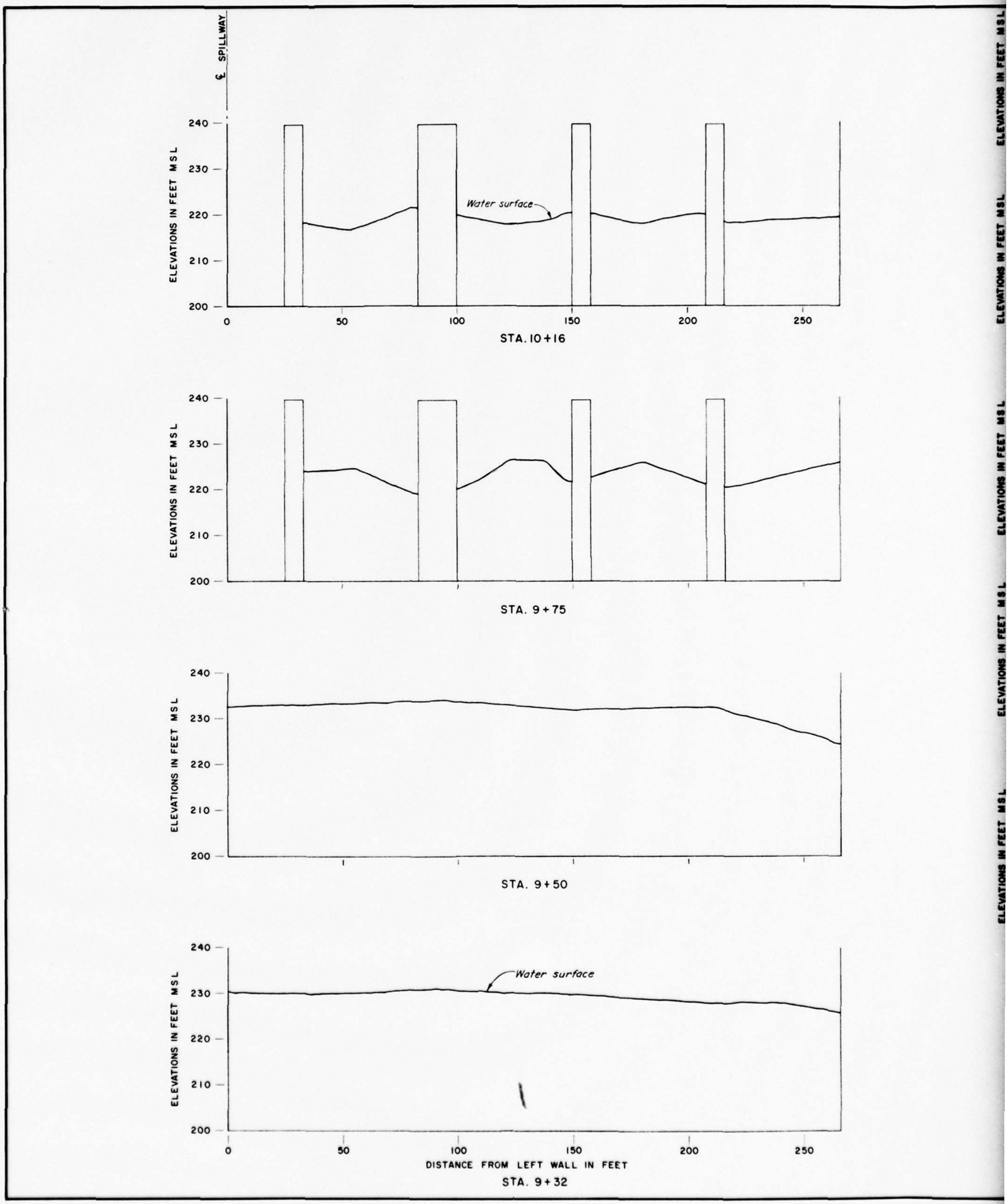
WHITTIER NARROWS-SAN GABRIEL SPILLWAY

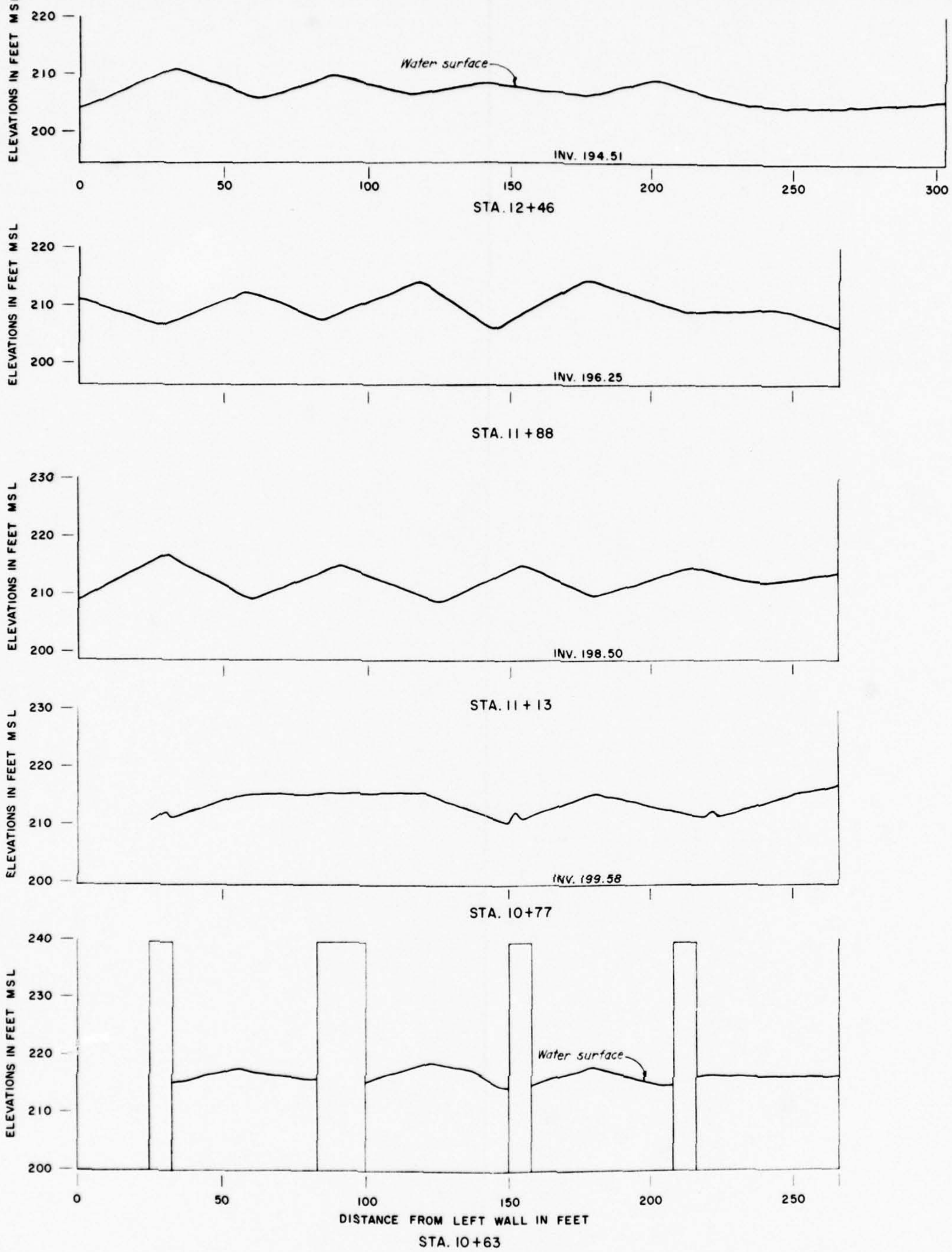
WATER DEPTH CONTOURS

HALF-SECTION

SPILLWAY PLAN F

CONTOUR INTERVAL = 1 FT


℄ SPILLWAY
ELEVATIONS IN FEET MSL
240
230
220
210
200
Water surface
0
50
100
150
200
250
STA. 10+16
ELEVATIONS IN FEET MSL
STA. 9+75
ELEVATIONS IN FEET MSL
STA. 9+50
ELEVATIONS IN FEET MSL
Water surface
DISTANCE FROM LEFT WALL IN FEET
STA. 9+32



ALL SECTIONS SHOWN LOOKING UPSTREAM
NOTE: SEE ALSO PLATE 9

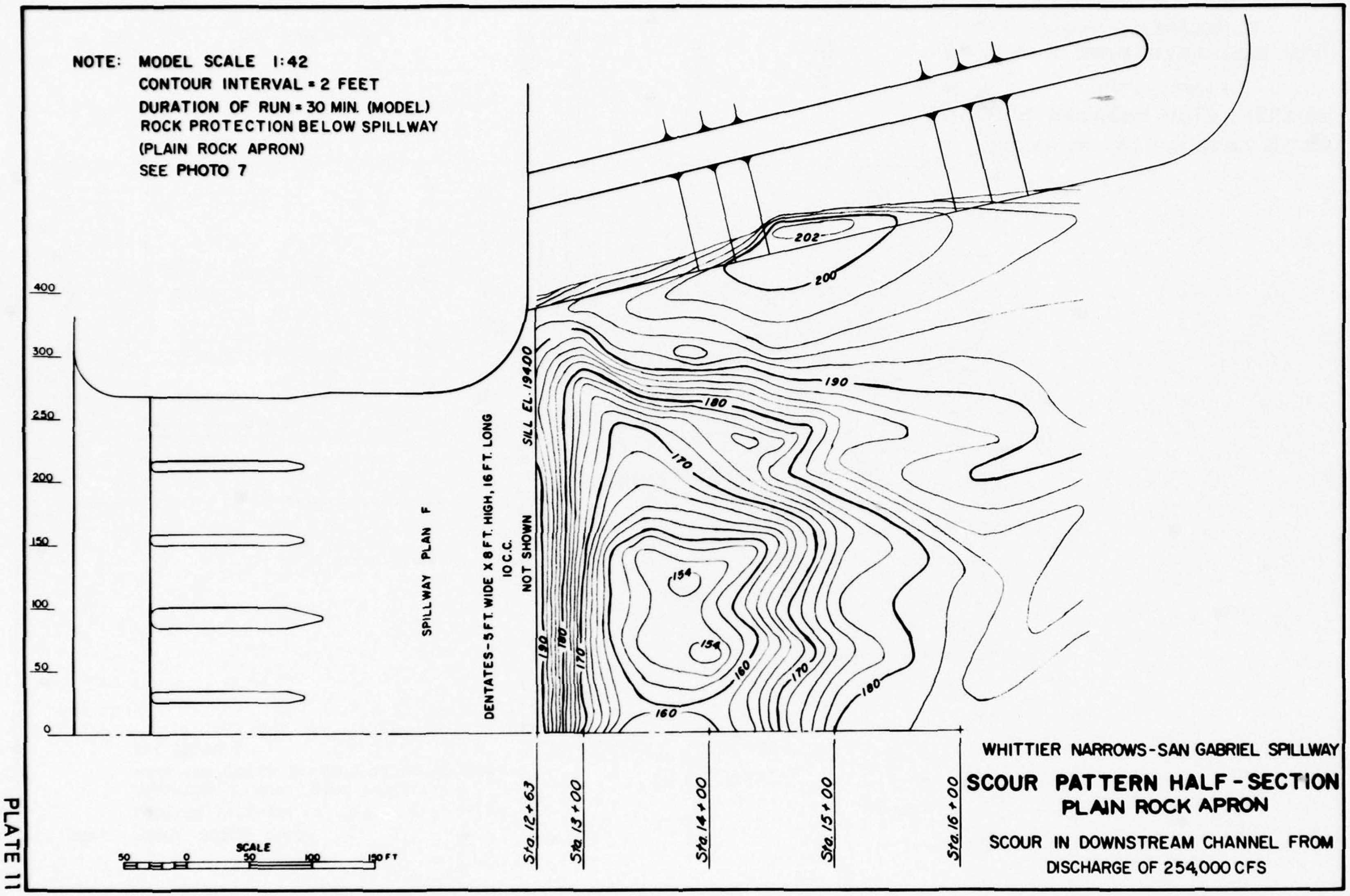
WHITTIER NARROWS-SAN GABRIEL SPILLWAY

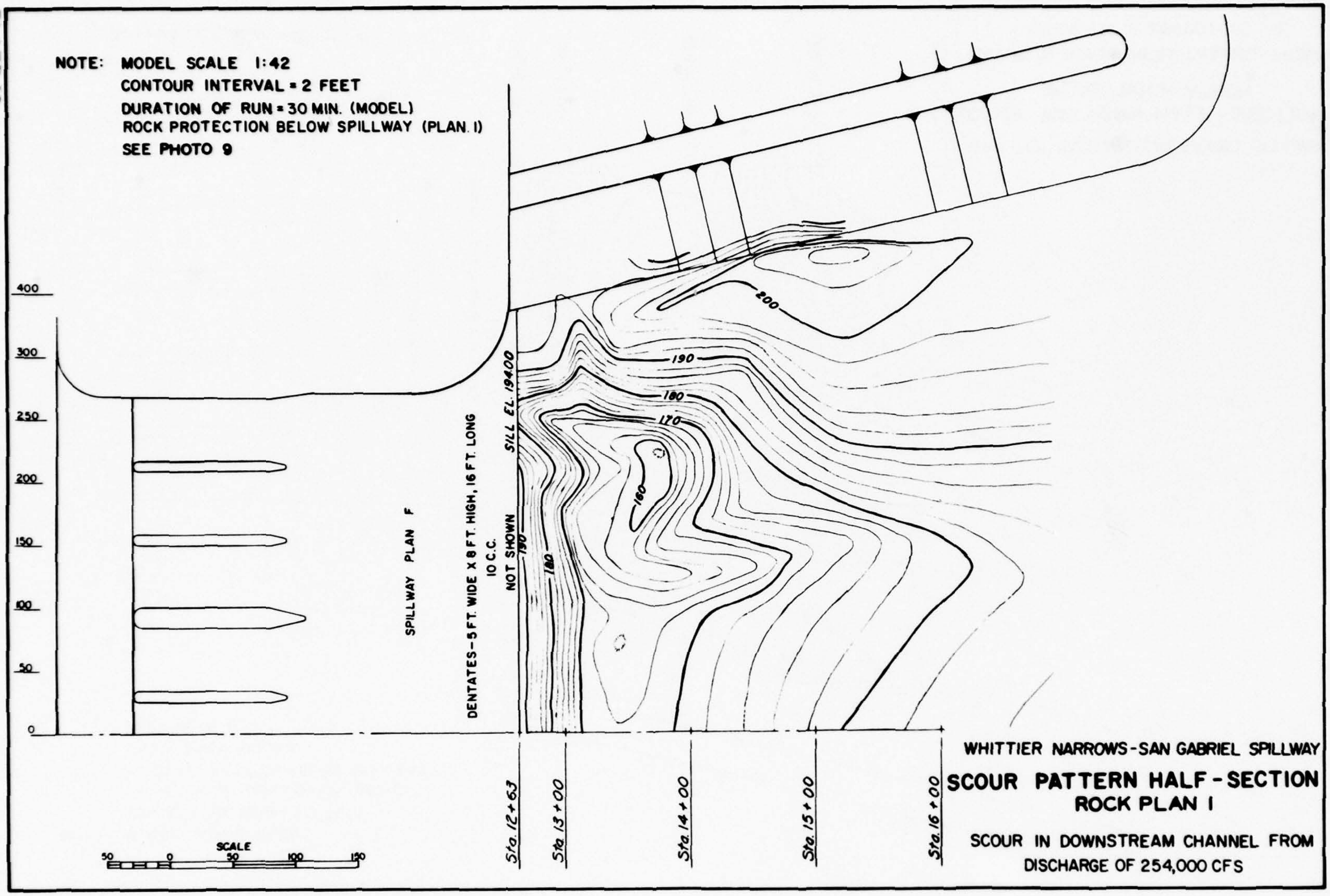
WATER-SURFACE CROSS SECTIONS

SPILLWAY PLAN F

HALF-SECTION

DISCHARGE 254,000 CFS

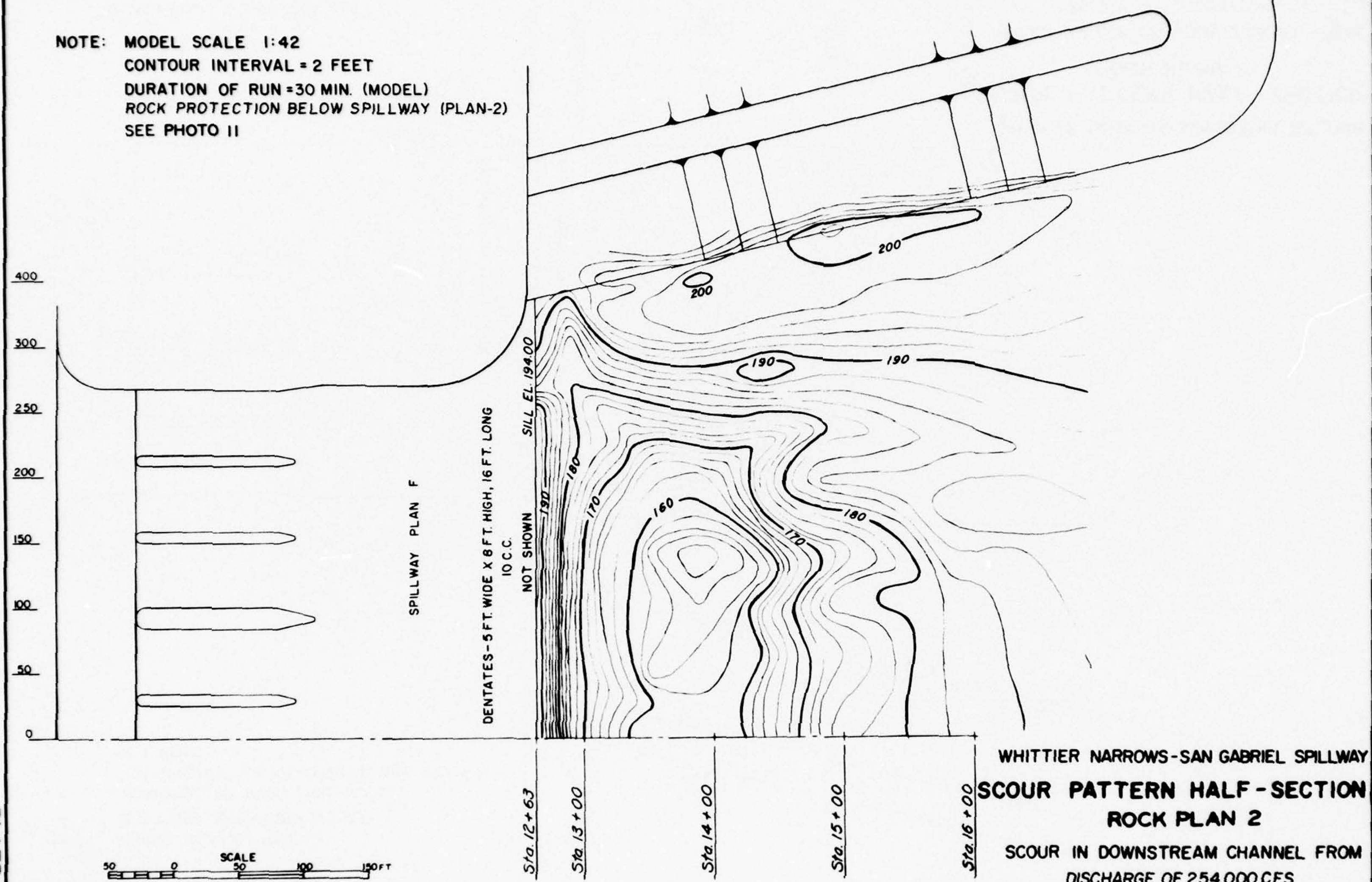

NOTE: MODEL SCALE 1:42
CONTOUR INTERVAL = 2 FEET
DURATION OF RUN = 30 MIN. (MODEL)
ROCK PROTECTION BELOW SPILLWAY
(PLAIN ROCK APRON)
SEE PHOTO 7
400
300
250
200
150
100
50
0
SPILLWAY PLAN F
DENTATES-5 FT. WIDE X 8 FT. HIGH, 16 FT. LONG
10 C.C.
NOT SHOWN
SILL EL. 194.00
202
200
190
180
170
154
154
160
170
180
160
190
180
170
Sta. 12+63
Sta. 13+00
Sta. 14+00
Sta. 15+00
Sta. 16+00
SCALE
50 0 50 100 150 FT
WHITTIER NARROWS-SAN GABRIEL SPILLWAY
SCOUR PATTERN HALF-SECTION
PLAIN ROCK APRON
SCOUR IN DOWNSTREAM CHANNEL FROM
DISCHARGE OF 254,000 CFS



NOTE: MODEL SCALE 1:42
CONTOUR INTERVAL = 2 FEET
DURATION OF RUN = 30 MIN. (MODEL)
ROCK PROTECTION BELOW SPILLWAY (PLAN. I)
SEE PHOTO 9
400
300
250
200
150
100
50
0
SPILLWAY PLAN F
DENTATES - 5 FT. WIDE X 8 FT. HIGH, 16 FT. LONG
10 C.C.
NOT SHOWN
SILL EL. 194.00
200
190
180
170
160
Sta. 12+63
Sta. 13+00
Sta. 14+00
Sta. 15+00
Sta. 16+00
SCALE
50
0
50
100
150
WHITTIER NARROWS-SAN GABRIEL SPILLWAY
SCOUR PATTERN HALF-SECTION
ROCK PLAN I
SCOUR IN DOWNSTREAM CHANNEL FROM
DISCHARGE OF 254,000 CFS

PLATE 12

NOTE: MODEL SCALE 1:42
CONTOUR INTERVAL = 2 FEET
DURATION OF RUN = 30 MIN. (MODEL)
ROCK PROTECTION BELOW SPILLWAY (PLAN-2)
SEE PHOTO 11

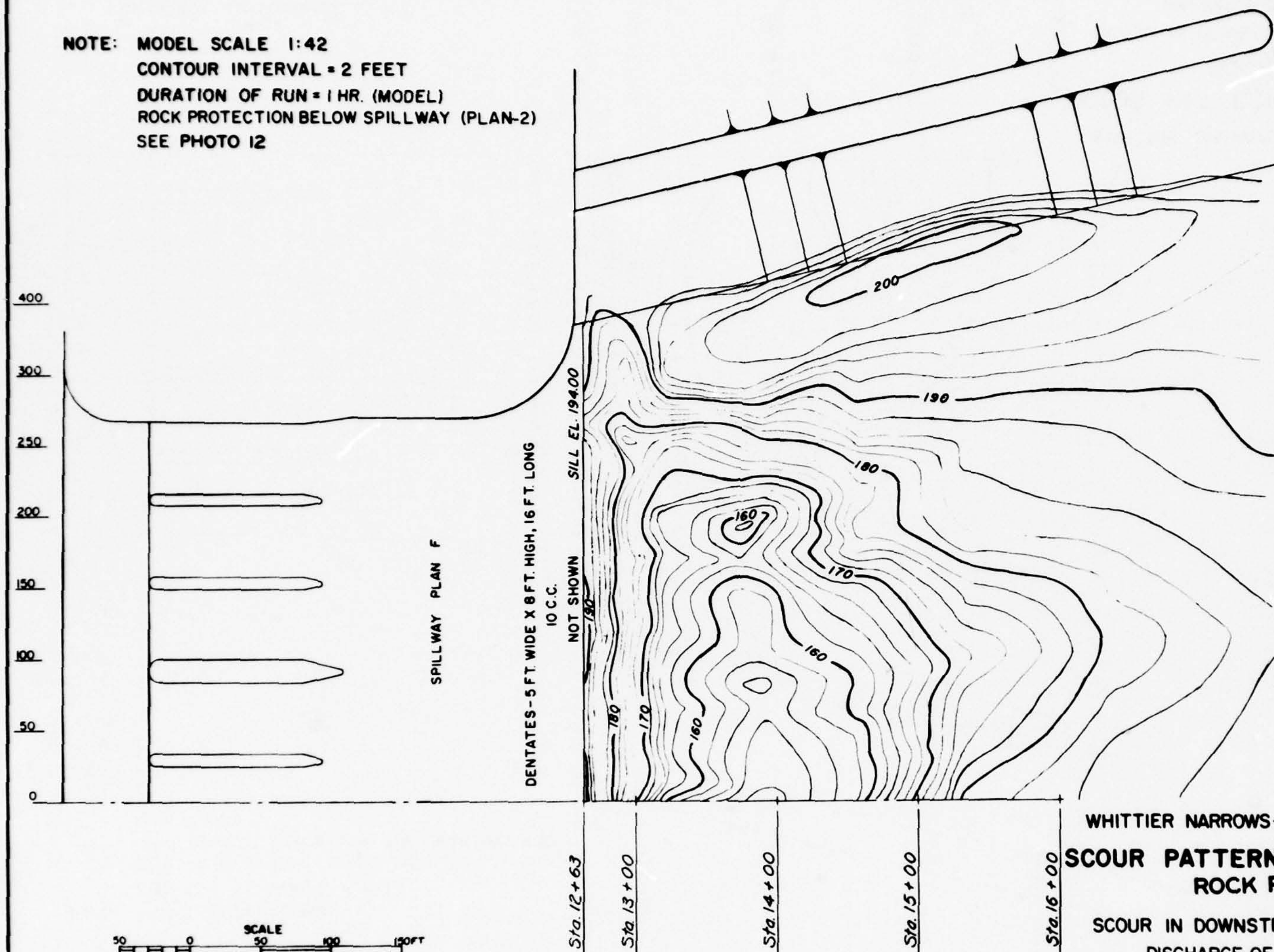


WHITTIER NARROWS-SAN GABRIEL SPILLWAY

SCOUR PATTERN HALF-SECTION
ROCK PLAN 2

SCOUR IN DOWNSTREAM CHANNEL FROM DISCHARGE OF 254,000 CFS

NOTE: MODEL SCALE 1:42
CONTOUR INTERVAL = 2 FEET
DURATION OF RUN = 1 HR. (MODEL)
ROCK PROTECTION BELOW SPILLWAY (PLAN-2)
SEE PHOTO 12

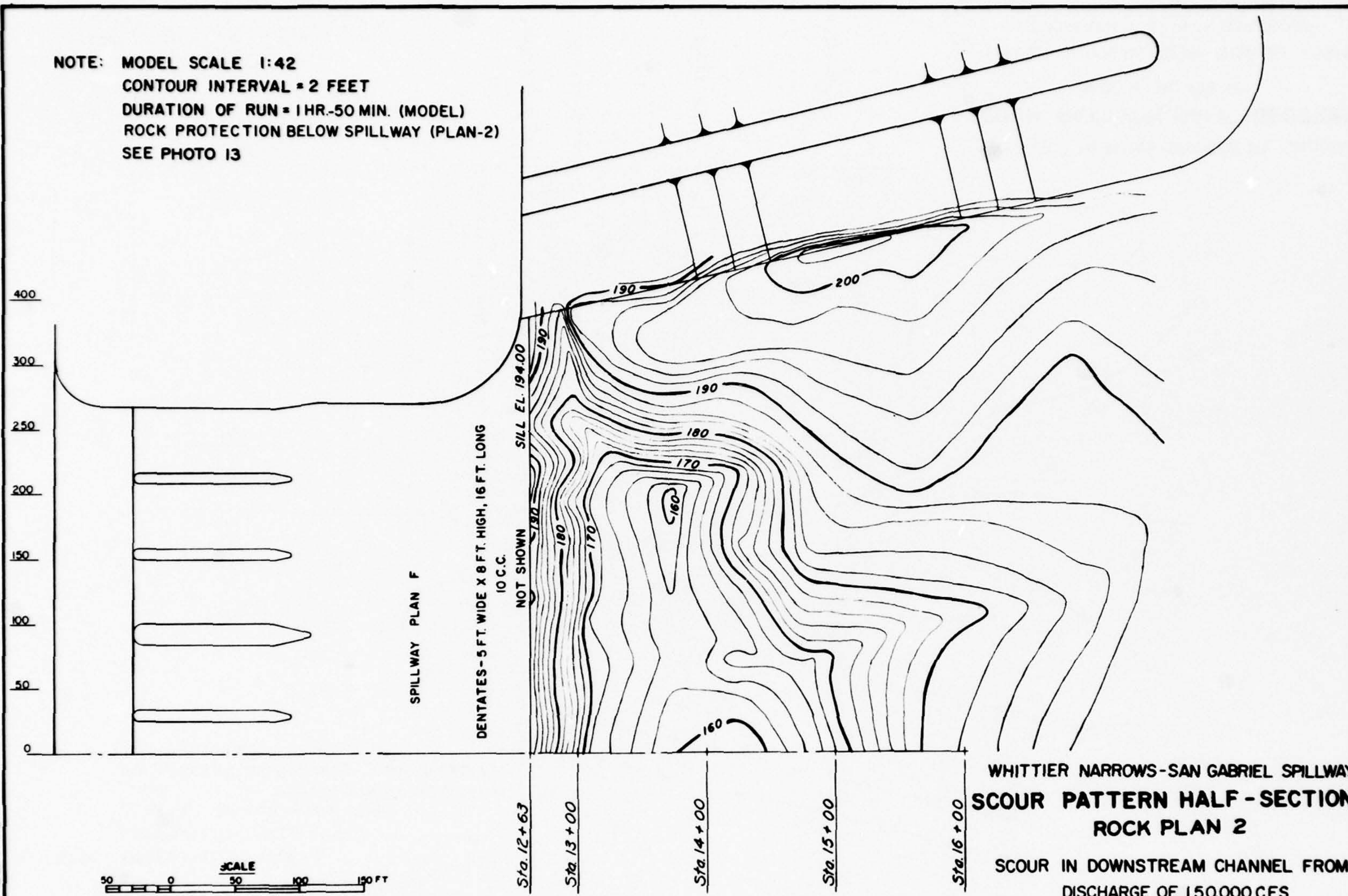


WHITTIER NARROWS-SAN GABRIEL SPILLWAY

SCOUR PATTERN HALF-SECTION
ROCK PLAN 2

SCOUR IN DOWNSTREAM CHANNEL FROM DISCHARGE OF 200,000 CFS

PLATE 14



WHITTIER NARROWS-SAN GABRIEL SPILLWAY

SCOUR PATTERN HALF-SECTION

ROCK PLAN 2

SCOUR IN DOWNSTREAM CHANNEL FROM DISCHARGE OF 150,000 CFS

NOTE: MODEL SCALE 1:42
CONTOUR INTERVAL = 2 FEET
DURATION OF RUN = 6 HRS. 40 MIN. (MODEL)
(COMPLETE HYDROGRAPH)
ROCK PROTECTION BELOW SPILLWAY (PLAN-2)
SEE PHOTO 14

WHITTIER NARROWS-SAN GABRIEL SPILLWAY
SCOUR PATTERN HALF-SECTION
ROCK PLAN 2
SCOUR IN DOWNSTREAM CHANNEL FROM
HYDROGRAPH RUN—PEAK 254,000 CFS

PLATE 16

PART III: RIO HONDO OUTLET STRUCTURE

Technical Features of Model Construction

Description of model

30. The model of the outlet structure, constructed to a scale ratio of 1:24, model to prototype, reproduced a portion of the earth-fill dam embankment on each side of the gate control section, an upstream approach section, and 1,200 ft of trapezoidal channel downstream from the outlet structure. A warped-wall transition 250 ft long connects the converging rectangular section downstream from the outlet structure to the trapezoidal channel. Part of the reservoir area was simulated by a wooden forebay equipped with baffles to ensure tranquil flow conditions. All portions of the model were constructed of wood, namely timbers and waterproofed plywood. The control section was divided into four bays. The prototype bays are 30 ft wide and separated by 8-ft-wide piers. The adjustable model radial gates were constructed of sheet metal. The longitudinal slopes of the Rio Hondo channel were 0.00859 between the outlet structure and sta 15+00 and about 0.0019 from sta 15+00 to the end of the model. Wooden rails set to grade along each side of the model provided a reference plane for use of all measuring devices.

Scale relations

31. The general relations for the transfer of model data to prototype equivalents or vice versa are presented below:

Dimension	Ratio	Scale Relations
Length	L_r	1:24
Area	$A_r = L_r^2$	1:576
Velocity	$V_r = L_r^{1/2}$	1:4.90
Time	$T_r = L_r^{1/2}$	1:4.90
Discharge	$Q_r = L_r^{5/2}$	1:2,822
Roughness coefficient	$N_r = L_r^{1/6}$	1:1.698

Procedures and Accomplishment of Outlet Model Tests

Procedure of tests

32. The general model testing procedure for the various features of the outlet structure design was as follows: (a) study flow conditions in the outlet works for an uncontrolled and controlled discharge of 40,000 cfs, (b) test various pier designs to obtain adequate drawdown from the approach channel to the gate section, and (c) determine the gate discharge characteristics of the outlet works for various combinations of gate openings and reservoir water-surface elevations.

Original design

33. This design, as shown in Plate 17 and Photo 16, incorporated the following: (a) an approach channel having parallel walls with 30-ft-radius curves at upstream toe of levee (no wing walls), (b) sloping 95-ft-long dividing piers upstream and vertical semicircular tails downstream, and (c) an exit channel (Rio Hondo channel) downstream from the outlet structure.

34. Tests simulating a prototype discharge of 40,000 cfs with all gates fully open and with all gates open 10 ft were made on the original design (Photos 17 and 18, respectively). These tests indicated that no real hydraulic advantage could be gained by providing sloping extensions upstream. The dividing piers upstream did not produce a smooth division of flow into the gate passages. An unbalanced flow existed in the gate section, setting up high waves in the transition downstream. However, flow conditions in the channel downstream from the transition were stable. Water-surface elevations were measured in the channel for a discharge of 40,000 cfs with all gates completely open and reservoir water-surface el 207.0 and with all gates open 10 ft and reservoir water-surface el 229.0. Depth contours determined from water-surface measurements are shown in Plates 18 and 19.

35. Changes to the original design were made by modifying the pier design in an attempt to produce the required drawdown at the entrance to the gates in order to have ample clearance under the curtain

wall for the design discharge of 40,000 cfs. Four tests on three pier designs were conducted in connection with finding a suitable pier design for the Rio Hondo outlet works. The hydraulic conditions for these tests were studied using the same discharge (40,000 cfs) and all gates were completely open. These pier designs are shown in Plate 20.

Test 1 (reservoir
water-surface el 207.2)

36. The upstream dividing pier extensions in the original design were eliminated and replaced with a parabolic nose so that each end of the pier was vertical. There was very little improvement over the original design. Flow conditions at the gate section were not favorable. Downstream flow was somewhat improved. Waves downstream from the outlet structure were not considered objectionable as they diminished rapidly downstream. Only visual observations were made.

Test 2 (reservoir
water-surface el 206.7)

37. In this test, the piers were not altered from the previous test but 26-ft-high (top el 210.0) quadrant wing walls with a radius of 30 ft were added to the entrance channel to obtain sufficient drawdown at the gate entrance. The curved wing walls made the conditions better to the extent that smoother flow was obtained in the approach channel but still did not produce enough drawdown at the gate entrance to clear the curtain wall. Uneven flow distribution was noted through the gate section. Downstream flow conditions were very similar to those in the previous test. Only visual observations were made.

Test 3 (reservoir
water-surface el 206.1)

38. Pier extensions, 94 ft long and 26 ft high, tapering from the width of the pier (8 ft) to a width of 1-1/2 ft at the upstream end were tested. The three piers were symmetrical and vertical on both ends. The drawdown at the gate section was down to el 200.0 (depth under curtain wall, 16 ft). Improvement of the flow through the gate section was noted. The uneven flow distribution across the gate openings encountered in test 2 was partly eliminated (Photo 19). Downstream depth

at sta 12+50 (beginning of transition) was about 13 ft. When discharge was increased to 46,000 cfs (reservoir water-surface el 208.2), the depth under the curtain wall was 17 ft and the depth at sta 12+50 was about 14 ft. Overall conditions were good.

Test 4 (reservoir water-surface el 206.2)

39. It was necessary to further reduce the depth of flow in the approach channel. The reduction was accomplished by increasing the cross-sectional area upstream from the gate section. Piers 1 and 3 were constructed unsymmetrical. The outward side of piers 1 and 3 was beveled inward and the inward side was parallel to the center line of the outlet structure. The center pier remained symmetrical. These unsymmetrical piers improved flow conditions at the gate section, and stable flow conditions downstream were obtained. The flow was generally uniform in depth downstream from the gates. These piers, as designed, proved to be satisfactory. Of the three pier designs described above, the tests indicated that the unsymmetrical piers would give the best hydraulic performance for the design discharge (Photo 20).

Final design

40. In the final design, the outlet structure walls were flared out and 30-ft-long downstream pier extensions were added to the symmetrical piers used in test 3. For plan, profile, and sections see Plate 5. The flared walls and the tapered piers gave the approach channel a much greater cross-sectional area than that at the gate entrance. This design produced the most satisfactory flow conditions and no further revision was considered necessary (Photo 21). The coefficients of discharge for various gate openings are listed in Table 4. The discharge curves for various gate combinations (gates open 12 and 16 ft) are shown in Plate 21. The discharge curves for partial gate openings (all gates) are shown in Plate 22. The discharge curve for free flow condition is represented by the formula $Q = CLH^{3/2}$. The discharge coefficient, determined from the model for $Q = 40{,}000$ cfs, was 2.90. Water-surface profiles for discharges of 47,000 and 40,000 cfs with gates fully open are shown in Plates 23 and 24, respectively. For plot of test data as

depth contours for discharges of 50,000 and 40,000 cfs see Plates 25 and 26, respectively.

Table 4

Rio Hondo Outlet Structure Gate Calibration

4 GATES-OPEN 2 FEET

AVERAGE H(FT)	CFS	$C=\frac{Q}{A\sqrt{2gH}}$
3.148	2822	0.825
4.468	3386	0.832
6.076	3951	0.831
8.020	4151	0.828
10.252	5080	0.823
12.400	5644	0.831
15.088	6208	0.830
17.956	6773	0.830
21.124	7337	0.829
24.186	7402	0.834
28.012	8486	0.830
31.660	9030	0.833
35.740	9595	0.833
39.760	10159	0.836
44.440	10724	0.826

4 GATES-OPEN 3 FEET

AVERAGE H(FT)	CFS	$C=\frac{Q}{A\sqrt{2gH}}$
5.892	5644	0.806
8.652	6773	0.796
11.796	7902	0.796
15.480	9030	0.794
19.464	10159	0.796
24.036	11288	0.797
29.256	12417	0.794
35.016	13546	0.792
40.620	14674	0.797
45.000	15521	0.800

4 GATES-OPEN 4 FEET

AVERAGE H(FT)	CFS	$C=\frac{Q}{A\sqrt{2gH}}$
7.688	8466	0.793
10.484	9877	0.792
13.712	11288	0.791
17.540	12699	0.787
21.488	14110	0.790
25.964	15521	0.791
30.968	16932	0.790
36.176	18343	0.791
42.224	19754	0.789

4 GATES-OPEN 6 FEET

AVERAGE H(FT)	CFS	$C=\frac{Q}{A\sqrt{2gH}}$
6.86	11288	0.746
8.69	12697	0.746
10.78	14110	0.743
13.01	15521	0.747
15.55	16932	0.744
18.00	18343	0.747
20.76	19754	0.752
23.71	21161	0.752

4 GATES-OPEN 8 FEET

AVERAGE H(FT)	DISCHARGE CFS	$C=\frac{Q}{A\sqrt{2gH}}$
7.90	16932	0.790
11.43	19754	0.759
15.21	22576	0.752
19.72	25398	0.742
23.73	28220	0.752
28.83	31042	0.750
34.47	33864	0.749
40.24	36686	0.752
43.34	38097	0.751

4 GATES-OPEN 10 FEET

AVERAGE H(FT)	DISCHARGE CFS	$C=\frac{Q}{A\sqrt{2gH}}$
5.78	19754	0.853
7.89	21165	0.783
9.72	22576	0.753
11.03	23987	0.751
12.54	25398	0.745
14.08	26809	0.742
15.64	28220	0.741
17.29	29631	0.740
18.87	31042	0.743
20.64	32453	0.742
22.39	33864	0.749
24.36	35275	0.743
26.45	36686	0.741
28.38	38097	0.743
30.14	39508	0.748
34.17	40919	0.728
36.93	42330	0.724

4 GATES-OPEN 12 FEET

AVERAGE H(FT)	DISCHARGE CFS	$C=\frac{Q}{A\sqrt{2gH}}$
7.28	25398	0.816
8.65	26809	0.790
9.98	28220	0.774
11.20	29631	0.766
12.43	31042	0.763
13.72	32453	0.756
15.12	33864	0.755
16.33	35275	0.756
17.78	36686	0.754
19.30	38097	0.751
20.82	39508	0.750
22.33	40919	0.750
23.90	42330	0.750

2 CENTER GATES OPEN 12FT 2 OUTER GATES CLOSED

AVERAGE	DISCHARGE CFS	$C=\frac{Q}{A\sqrt{2gH}}$
8.79	14110	0.826
11.26	15521	0.801
13.52	16932	0.798
16.28	18343	0.787
18.56	19754	0.794
21.81	21165	0.785
24.77	22576	0.786
27.85	23987	0.787
31.04	25398	0.790
34.40	26809	0.792
37.70	28220	0.796

2 CENTER GATES OPEN 18FT 2 OUTER GATES CLOSED

AVERAGE	DISCHARGE CFS	$C=\frac{Q}{A\sqrt{2gH}}$
11.33	21165	0.817
13.51	22576	0.798
15.43	23987	0.794
17.40	25398	0.791
19.40	26809	0.790
21.63	28220	0.788
23.80	29631	0.789
25.98	31042	0.791
28.43	32453	0.791
31.15	33864	0.788

2 OUTER GATES OPEN 12FT 2 CENTER GATES CLOSED

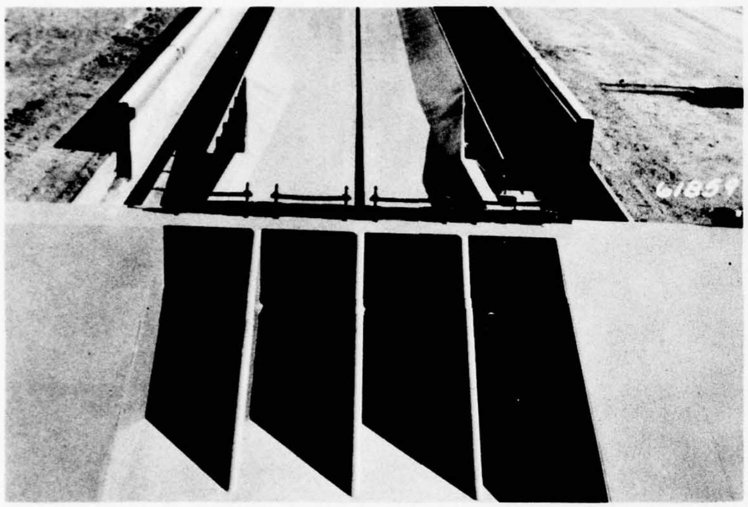
AVERAGE	DISCHARGE CFS	$C=\frac{Q}{A\sqrt{2gH}}$
4.506	11288	0.922
7.308	12699	0.813
9.976	14110	0.774
12.658	15521	0.765
15.256	16932	0.751
17.728	18343	0.755
21.064	19754	0.746
24.048	21165	0.746
27.228	22576	0.748
30.528	23987	0.751
34.184	25398	0.752
37.956	26809	0.754

2 OUTER GATES OPEN 18FT 2 CENTER GATES CLOSED

AVERAGE	DISCHARGE CFS	$C=\frac{Q}{A\sqrt{2gH}}$
9.522	19754	0.832
11.305	21165	0.818
13.308	22576	0.804
15.606	23987	0.789
17.580	25398	0.787
20.070	26809	0.777
22.602	28220	0.771
25.110	29631	0.768
27.678	31042	0.767
30.144	32453	0.768
32.982	33864	0.766

ALL 4 GATES OPEN FOR FREE FLOW CONDITION

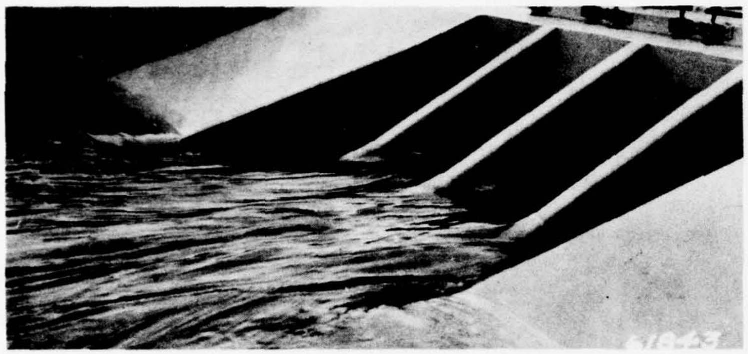
AVERAGE DEPTH FT	DISCHARGE CFS
8.842	8466
9.612	9877
10.468	11288
11.248	12699
11.964	14110
12.658	15521
13.390	16932
14.004	18343
14.892	19754
15.540	21165
16.132	22576
16.810	23987
17.350	25398
17.948	26809
18.580	28220
19.012	29631
19.528	31042
20.200	32453
20.662	33864
21.328	35275
21.849	36686
22.300	38097
22.744	39508
23.316	40919



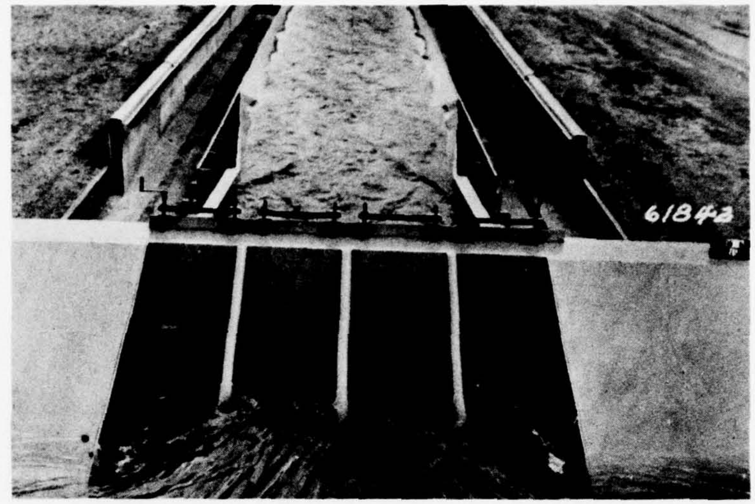
General view looking downstream. Note warped transition downstream of outlet works

Looking downstream

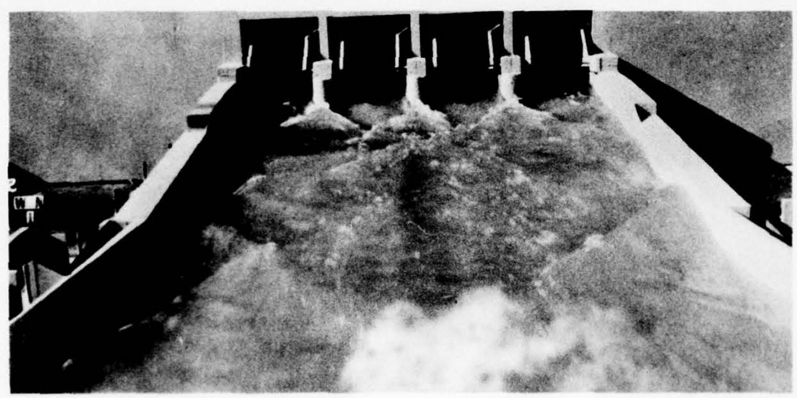
Photo 16. Original design



Side view of approach channel

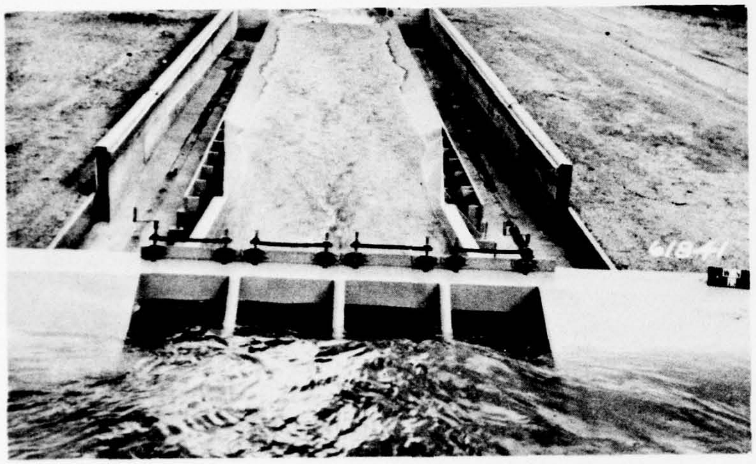


Looking downstream

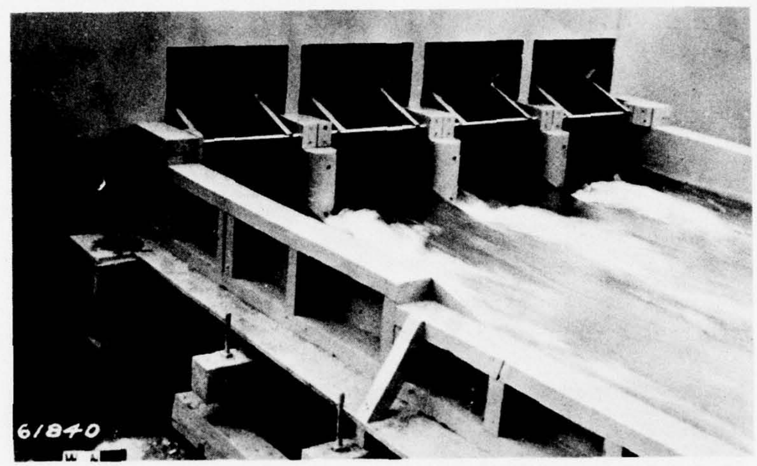


Looking upstream

Photo 17. Original design; discharge 40,000 cfs, gates fully open, reservoir water-surface el 207.0

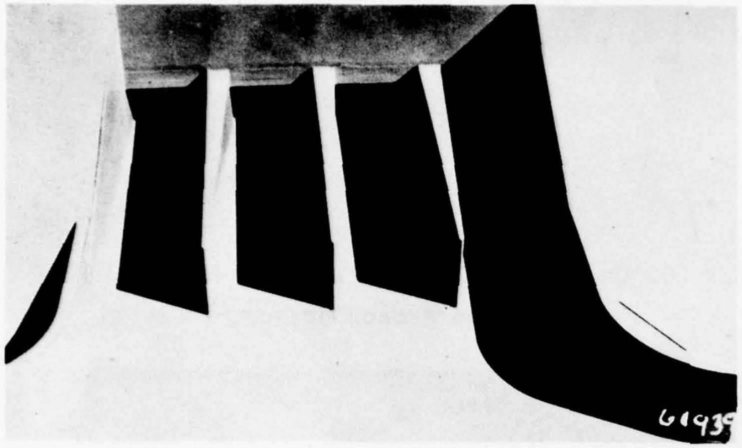


Looking downstream



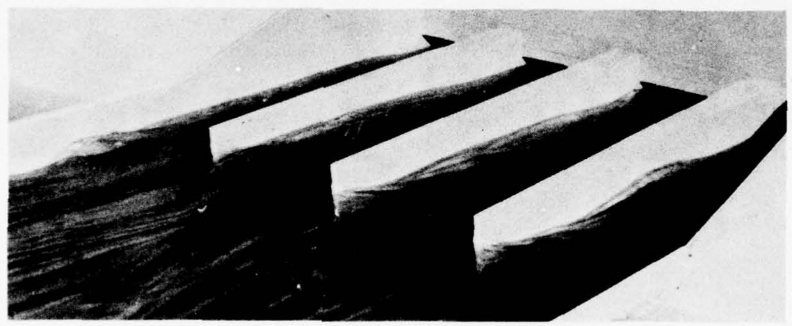
Looking upstream

Photo 18. Original design; discharge 40,000 cfs, gates open 10 ft, reservoir water-surface el 229.0



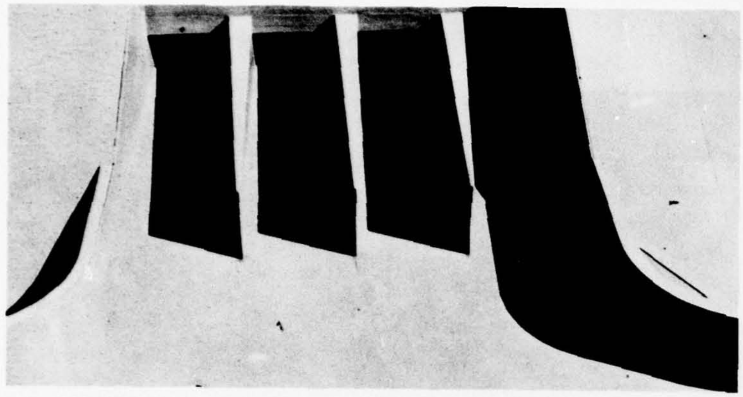
Looking downstream

Looking downstream, discharge 40,000 cfs, reservoir water-surface el 206.1

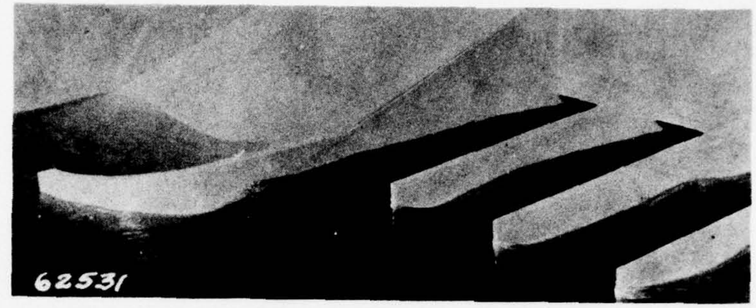


Side view, looking downstream

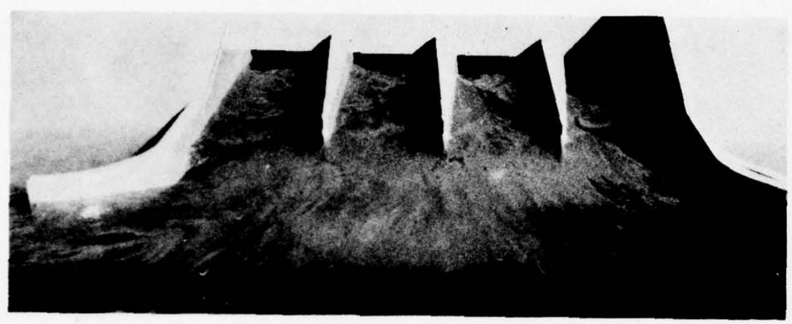
Photo 19. Test 3, all piers symmetrical. Top of wing walls at el 210.0



Looking downstream

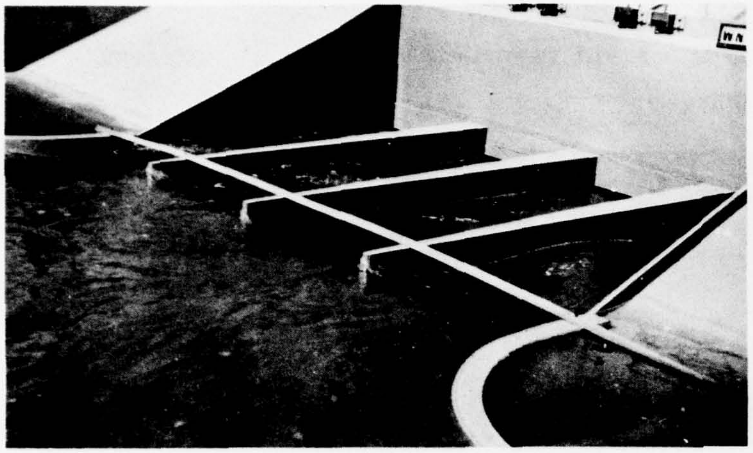


Side view, looking downstream, discharge 40,000 cfs, reservoir water-surface el 206.2

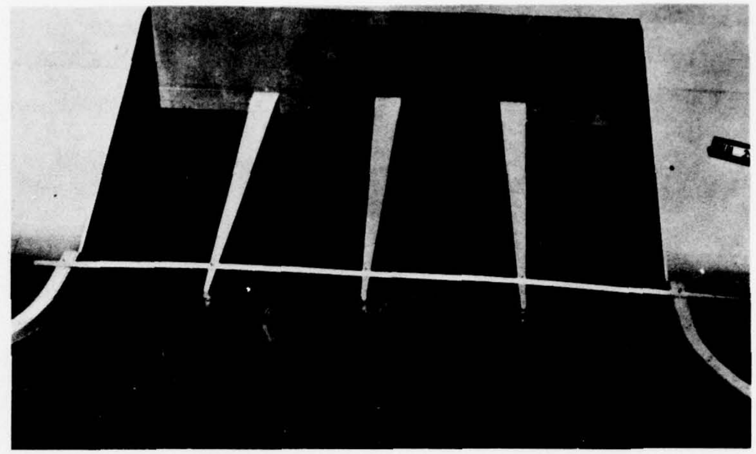


Looking downstream

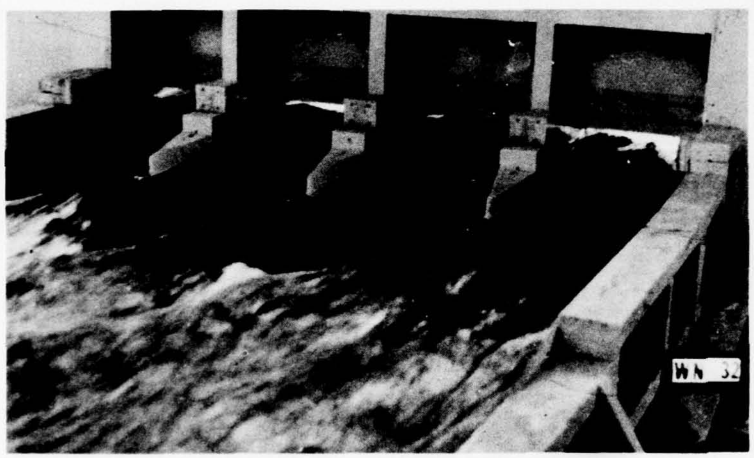
Photo 20. Test 4, unsymmetrical piers. Middle pier tapered, outside piers beveled inward. Top of wing wall at el 210.0

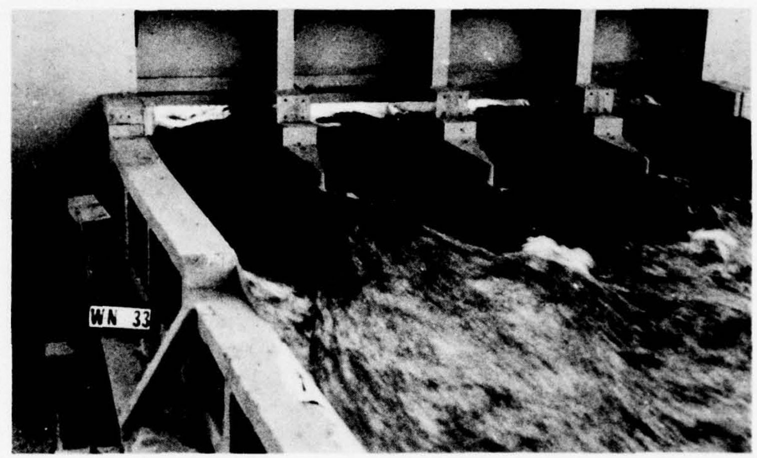


Side view from upstream



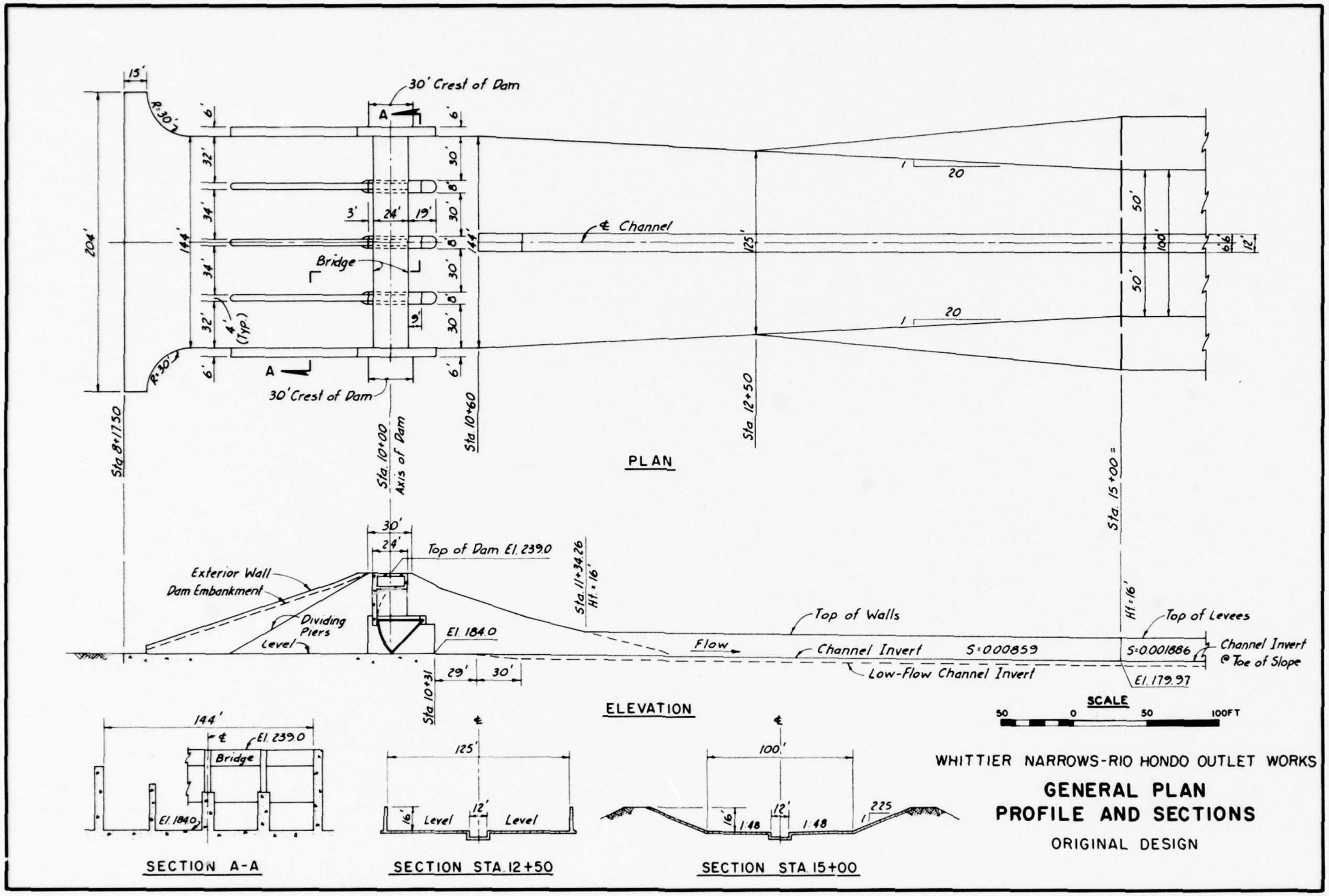
Looking downstream

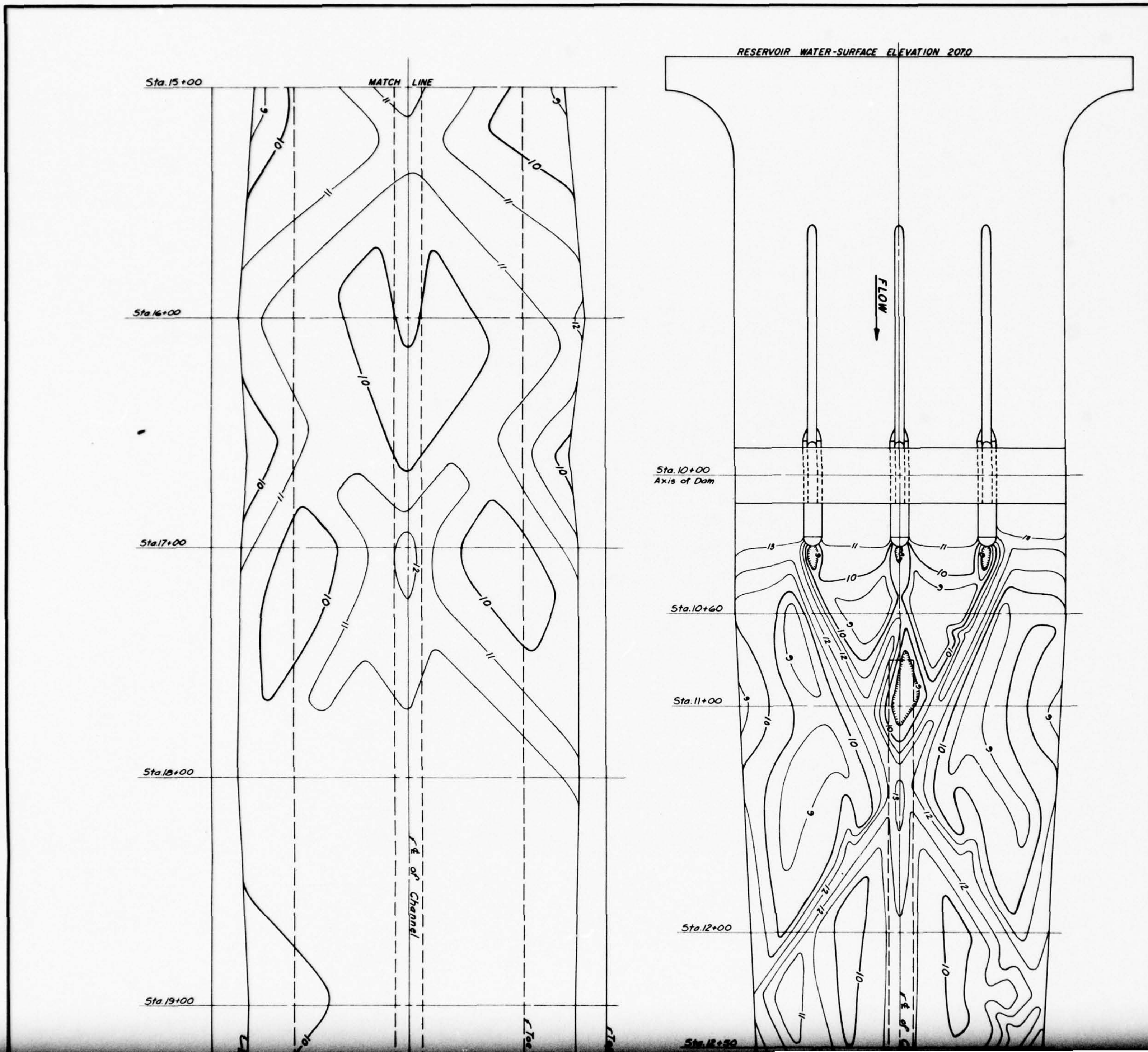


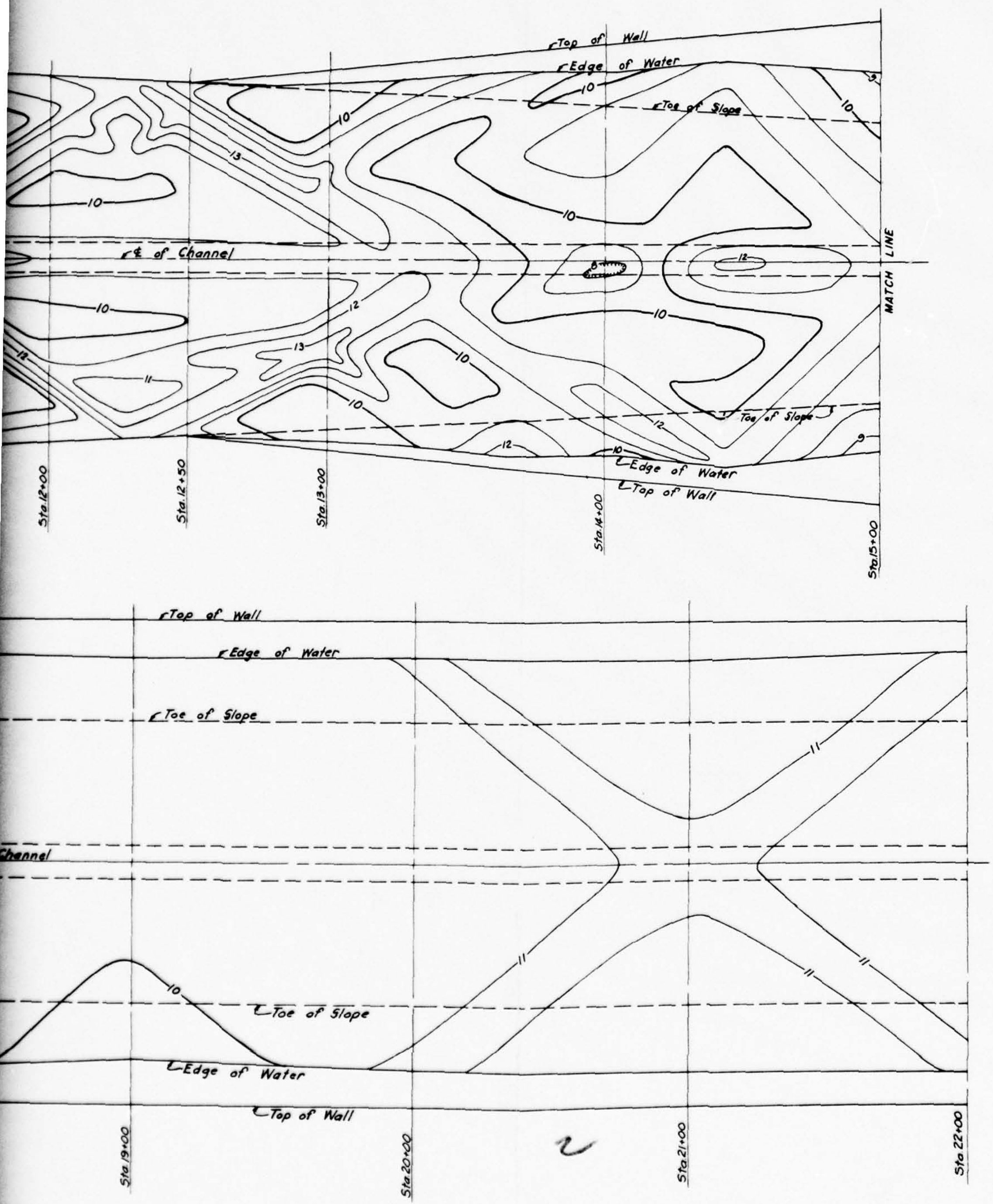


Looking upstream

Photo 21. Final design, discharge 47,000 cfs


30' Crest of Dam
15'
R=30'
204'
144'
6'
32'
34'
34'
32'
6'
4' (Typ.)
3'
24'
19'
19'
30'
8'
Bridge
A
Sta. 8+17.50
Sta. 10+00 Axis of Dam
Sta. 10+60
Sta. 12+50
Sta. 15+00 =
℄ Channel
125'
1
20
50'
50'
100'
6.6
12'
PLAN
30'
24'
Top of Dam El. 239.0
Exterior Wall
Dam Embankment
Dividing Piers
Level
El. 184.0
Sta. 10+31
29'
30'
Sta. 11+34.26
Ht. = 16'
Top of Walls
Flow
Channel Invert
S=0.00859
Low-Flow Channel Invert
Ht.=16'
Top of Levees
S=0.001886
Channel Invert @ Toe of Slope
El. 179.97
ELEVATION
SCALE
50
0
50
100 FT
144'
℄
El. 239.0
Bridge
El. 184.0
SECTION A-A
125'
16'
Level
12'
Level
SECTION STA. 12+50
100'
16'
1:48
12'
1:48
1
2.25
SECTION STA. 15+00
WHITTIER NARROWS-RIO HONDO OUTLET WORKS
GENERAL PLAN
PROFILE AND SECTIONS
ORIGINAL DESIGN


RESERVOIR WATER-SURFACE ELEVATION 207.0
FLOW
Sta. 10+00
Axis of Dam
Sta. 10+60
Sta. 11+00
Sta. 12+00
Sta. 12+50
MATCH LINE
Sta. 15+00
Sta. 16+00
Sta. 17+00
Sta. 18+00
Sta. 19+00
℄ of Channel
Toe

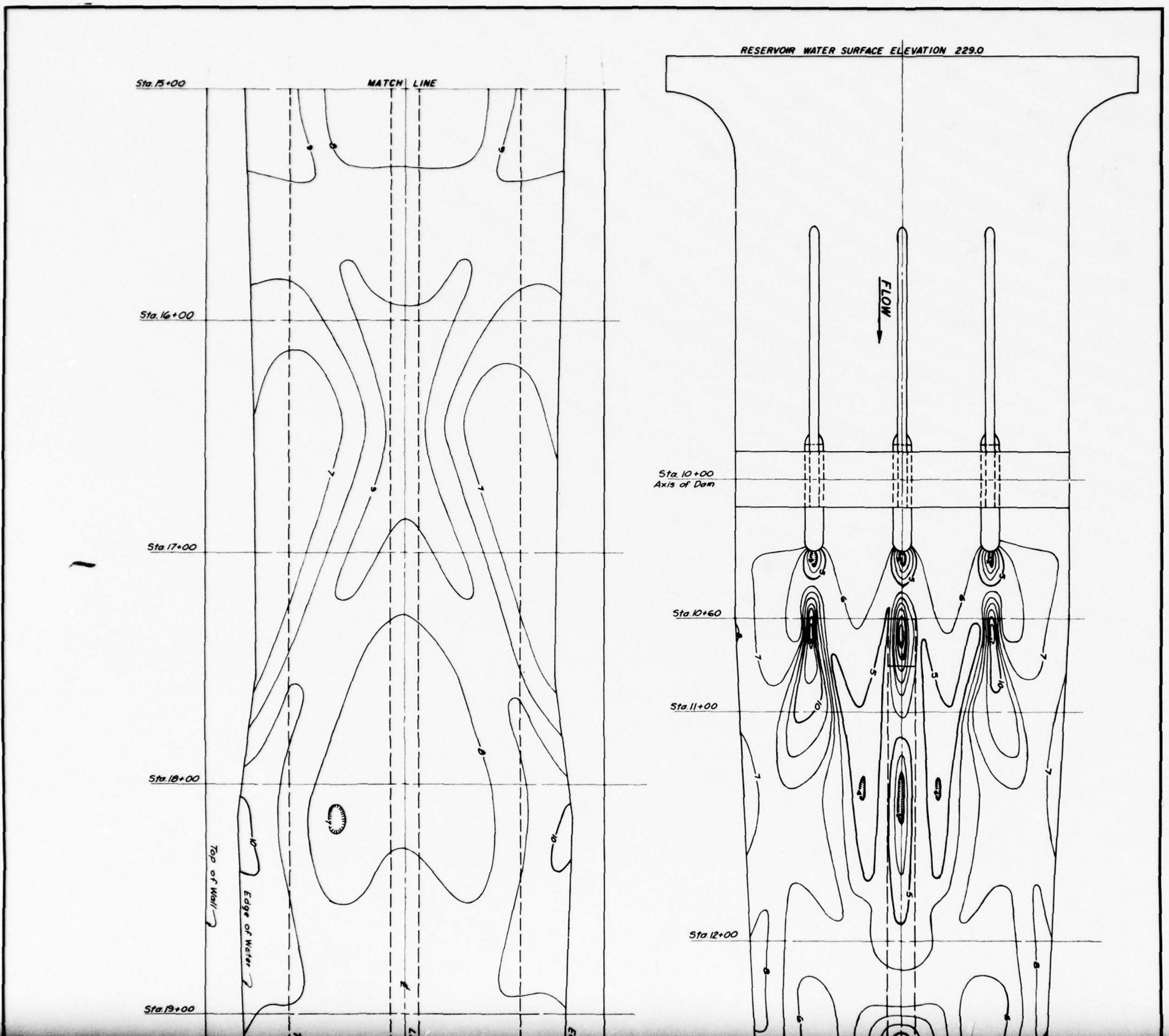


WHITTIER NARROWS-RIO HONDO OUTLET WORKS

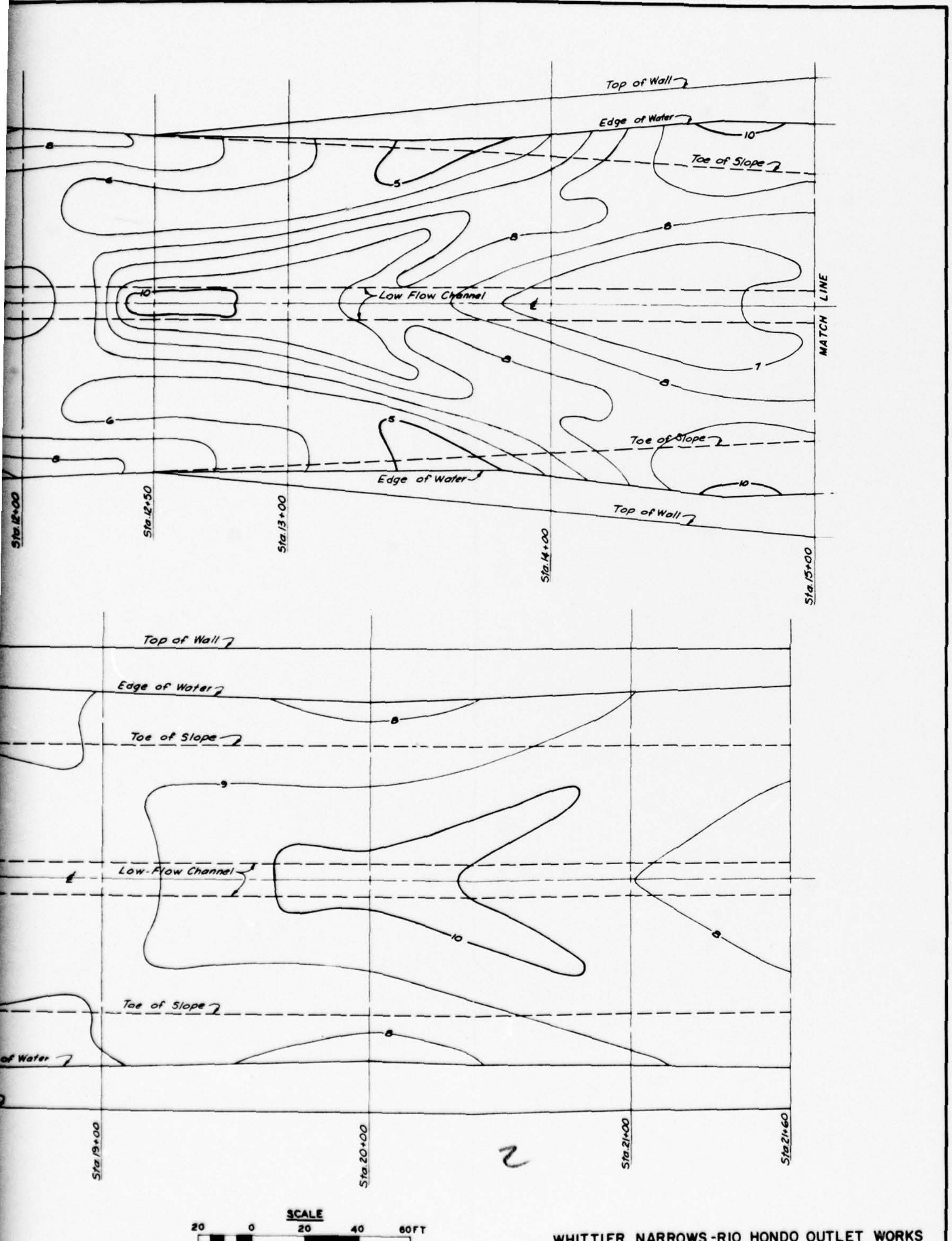
DEPTH CONTOURS

ORIGINAL DESIGN

DISCHARGE: 40,000 CFS; ALL GATES FULLY OPEN



RESERVOIR WATER SURFACE ELEVATION 229.0
FLOW
Sta. 10+00
Axis of Dam
Sta. 10+60
Sta. 11+00
Sta. 12+00
MATCH LINE
Sta. 15+00
Sta. 16+00
Sta. 17+00
Sta. 18+00
Sta. 19+00
Edge of Water
Top of Wall



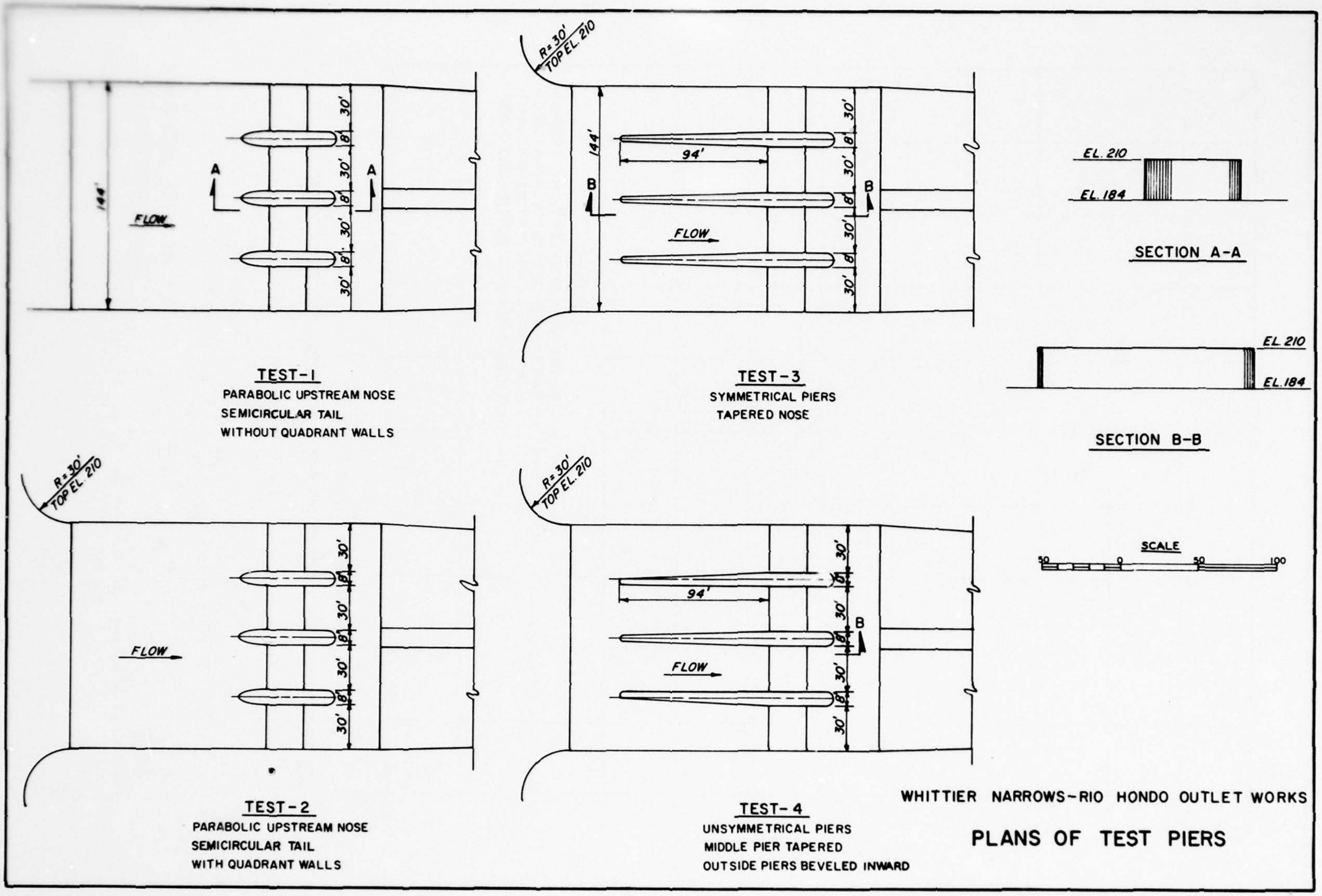
WHITTIER NARROWS-RIO HONDO OUTLET WORKS

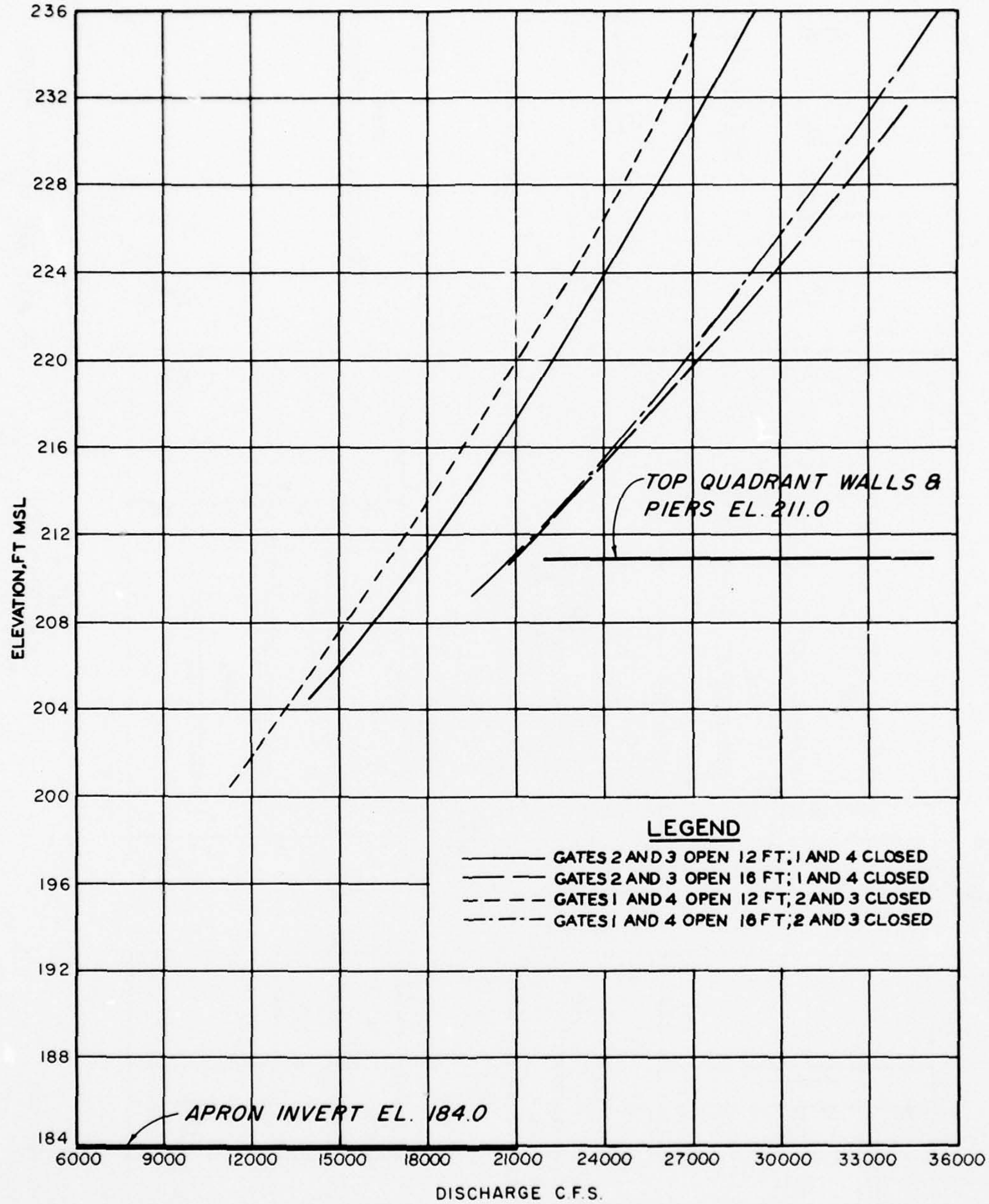
DEPTH CONTOURS

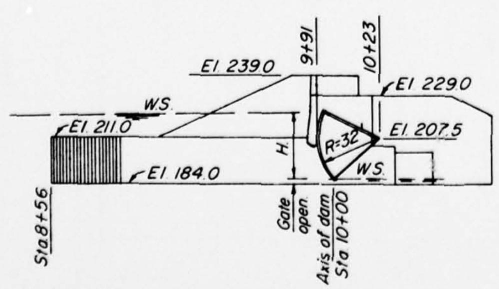
ORIGINAL DESIGN

DISCHARGE: 40,000 CFS; GATE OPENING 10.0 FT

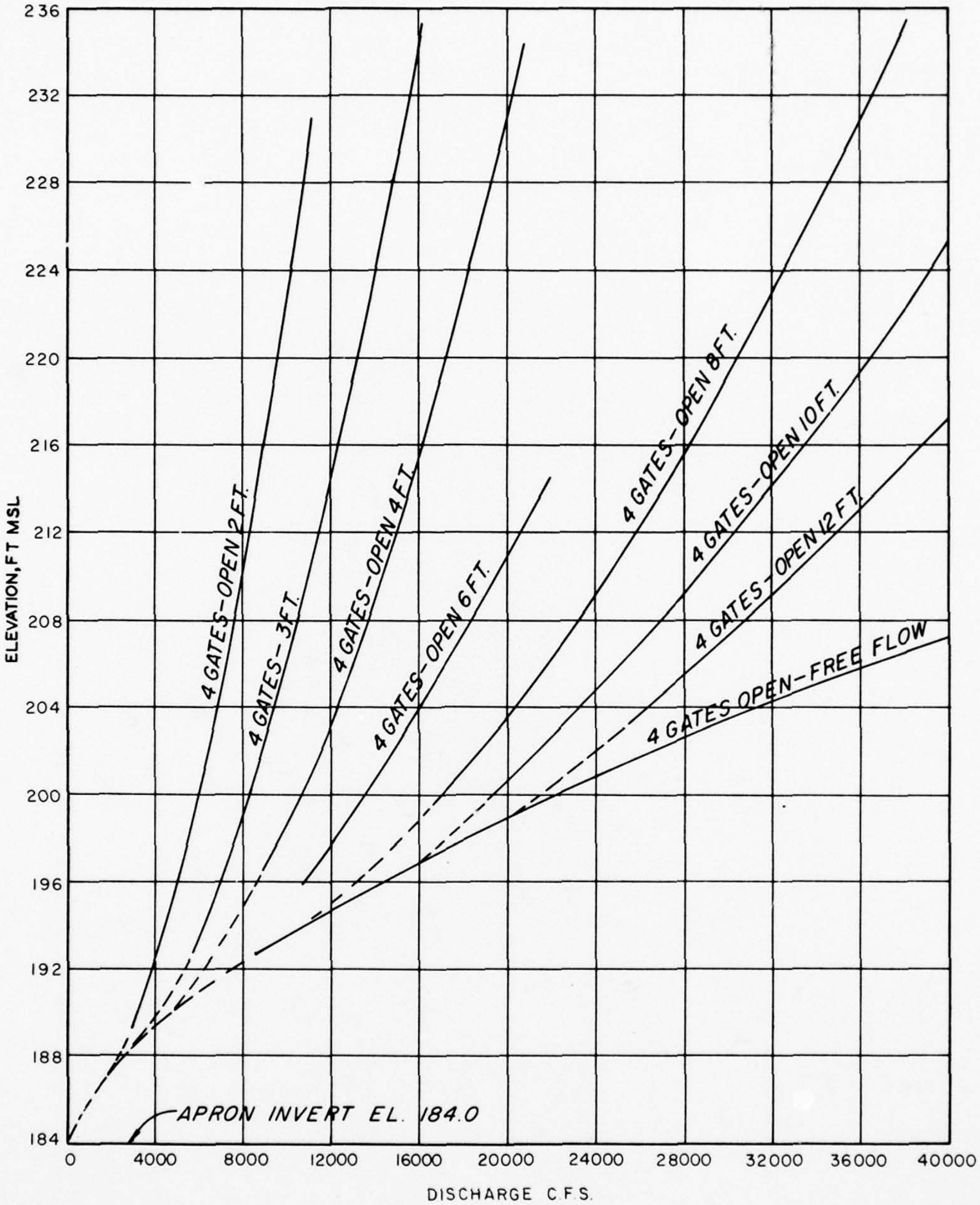
PLATE 19

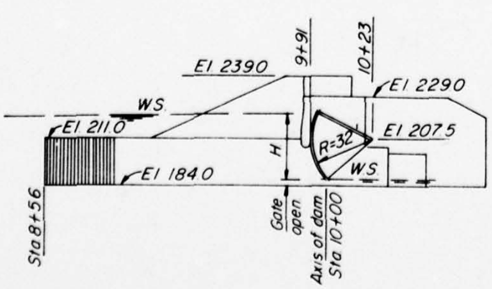

R=30'
TOP EL. 210
144'
FLOW
A
A
30'
8'
30'
8'
30'
8'
30'
TEST-1
PARABOLIC UPSTREAM NOSE
SEMICIRCULAR TAIL
WITHOUT QUADRANT WALLS
144'
94'
B
B
FLOW
TEST-3
SYMMETRICAL PIERS
TAPERED NOSE
EL. 210
EL. 184
SECTION A-A
EL. 210
EL. 184
SECTION B-B
SCALE
50
0
50
100
FLOW
TEST-2
PARABOLIC UPSTREAM NOSE
SEMICIRCULAR TAIL
WITH QUADRANT WALLS
94'
B
FLOW
TEST-4
UNSYMMETRICAL PIERS
MIDDLE PIER TAPERED
OUTSIDE PIERS BEVELED INWARD
WHITTIER NARROWS-RIO HONDO OUTLET WORKS
PLANS OF TEST PIERS



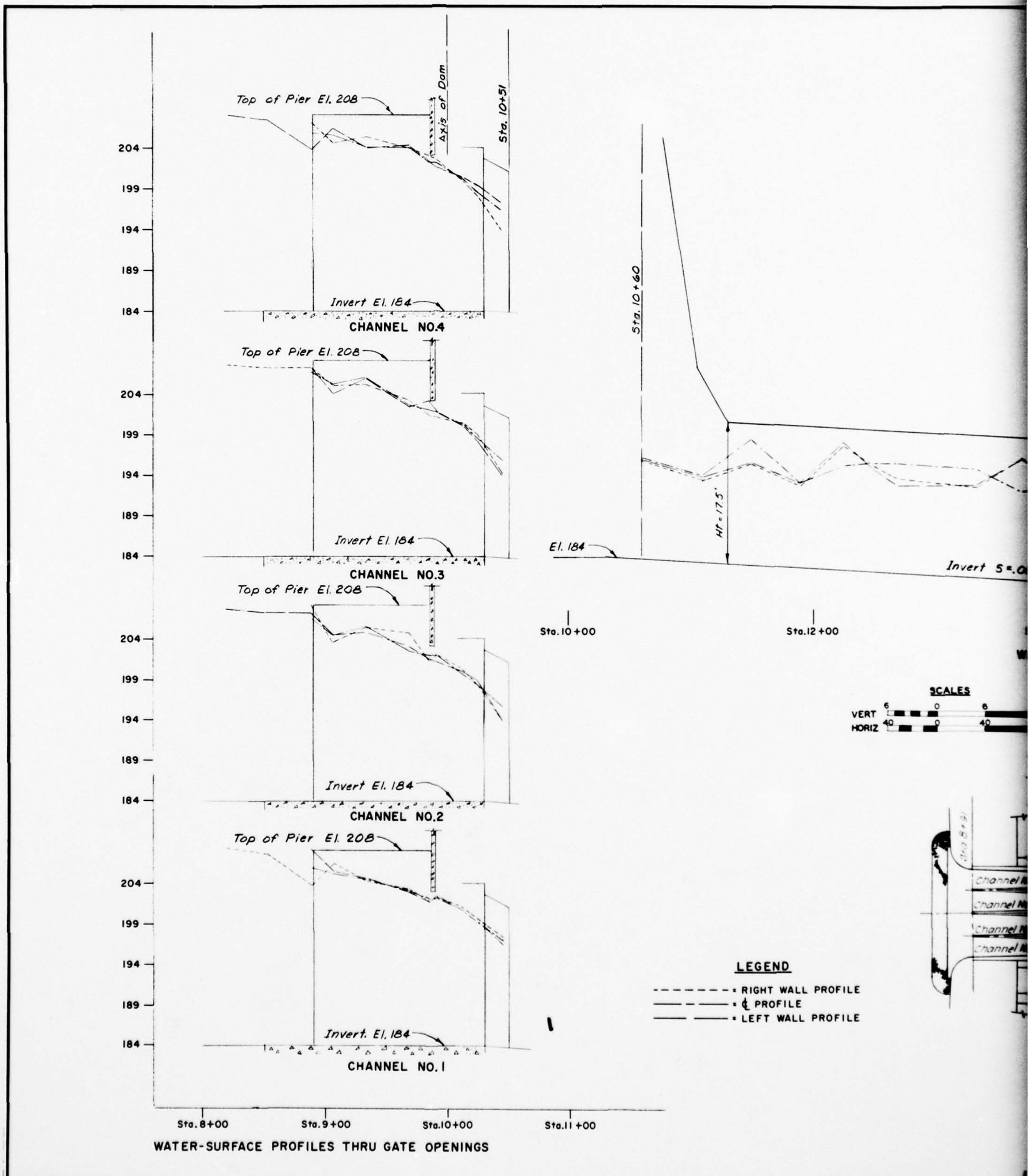


WHITTIER NARROWS-
RIO HONDO OUTLET
GATE CALIBRATION
FINAL DESIGN

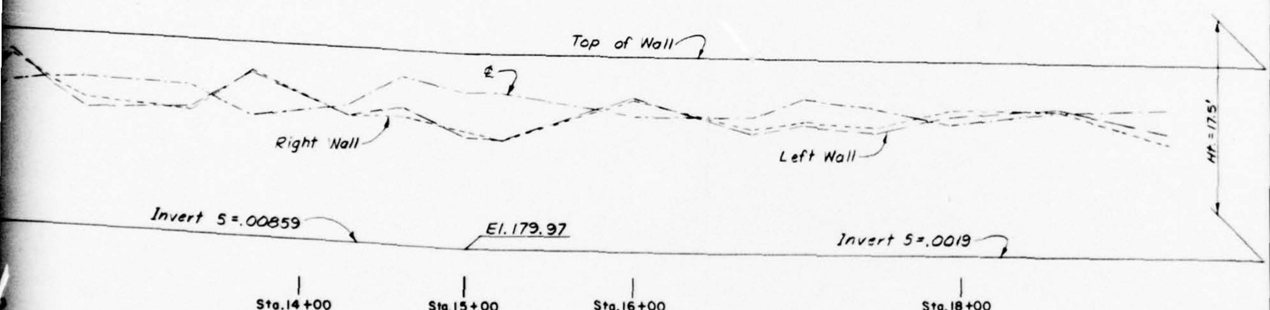




WHITTIER NARROWS-
RIO HONDO OUTLET
GATE CALIBRATION
FOUR GATES OPERATING
FINAL DESIGN

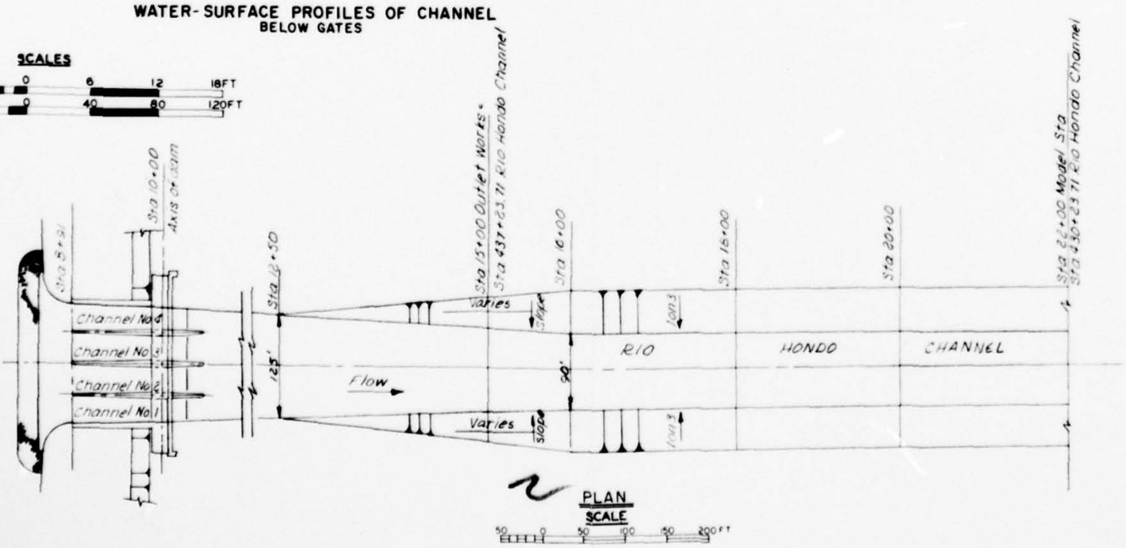


WATER-SURFACE PROFILES THRU GATE OPENINGS



WATER-SURFACE PROFILES OF CHANNEL
BELOW GATES

SCALES
VERT 6 0 6 12 18FT
HORIZ 40 0 40 80 120FT



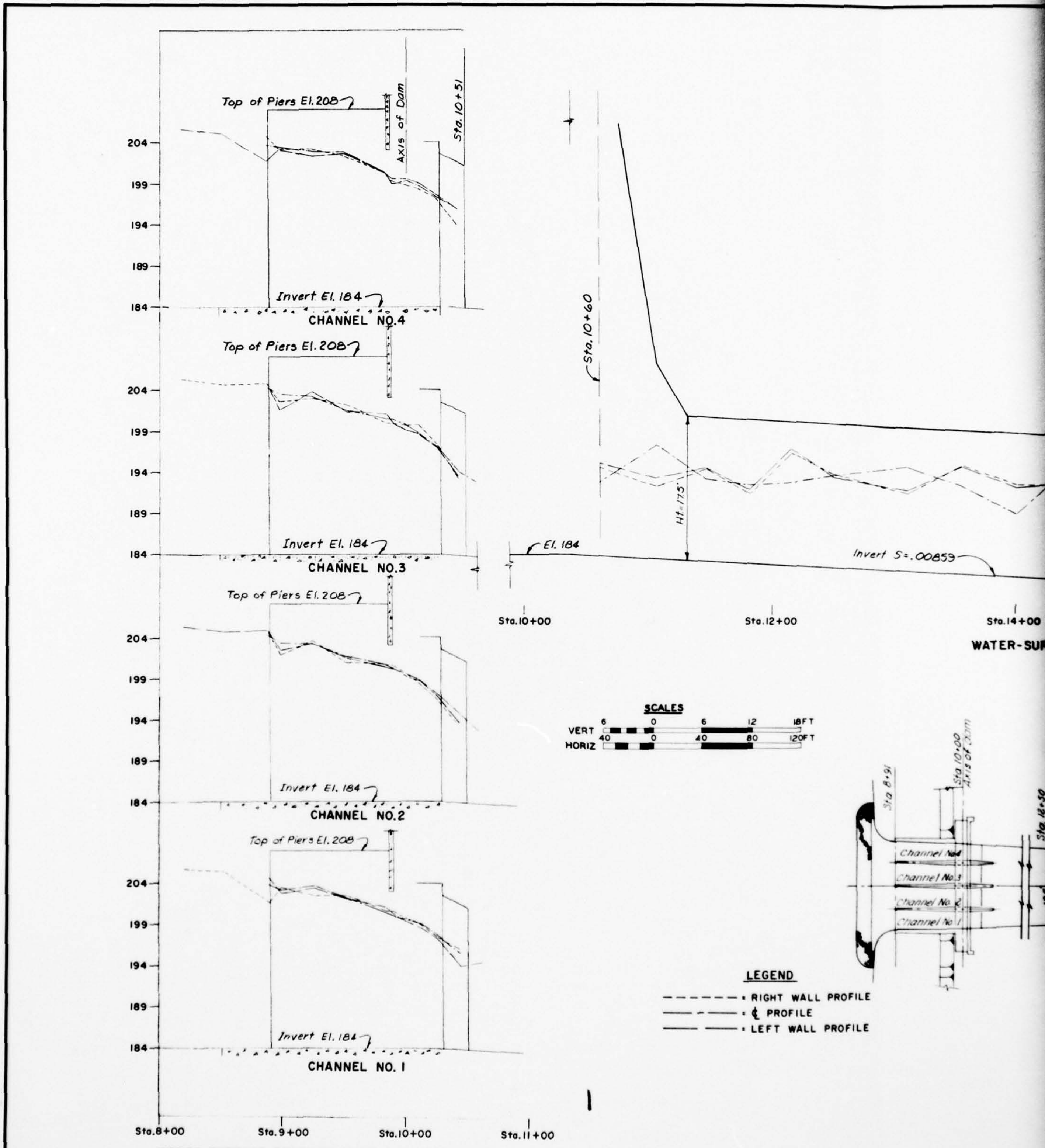
PLAN
SCALE
50 0 50 100 150 200FT

WHITTIER NARROWS – RIO HONDO OUTLET AND CHANNEL

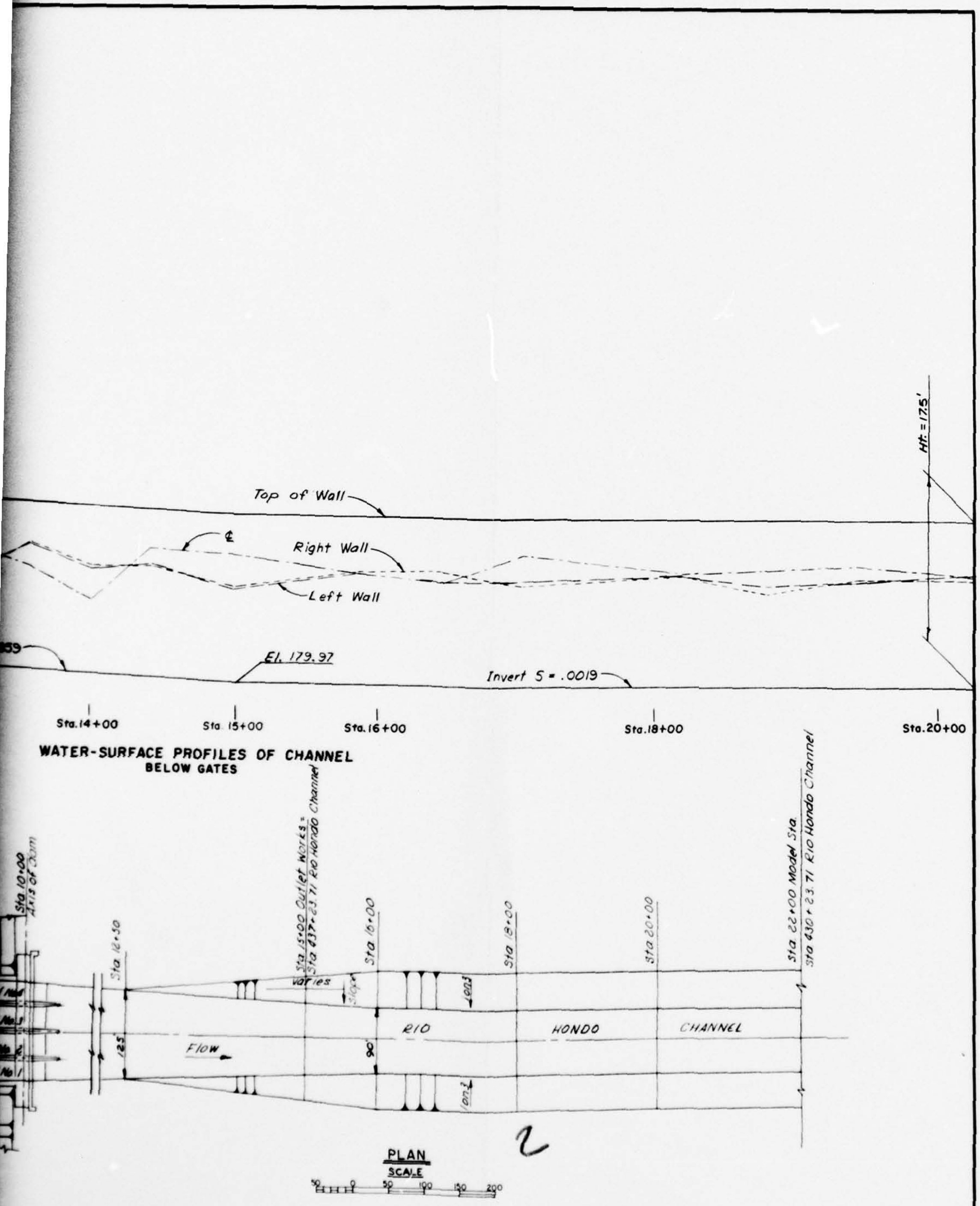
WATER-SURFACE PROFILES

DISCHARGE: 47,000 CFS; GATES OPEN

FINAL DESIGN



WATER-SURFACE PROFILES THRU GATE OPENINGS

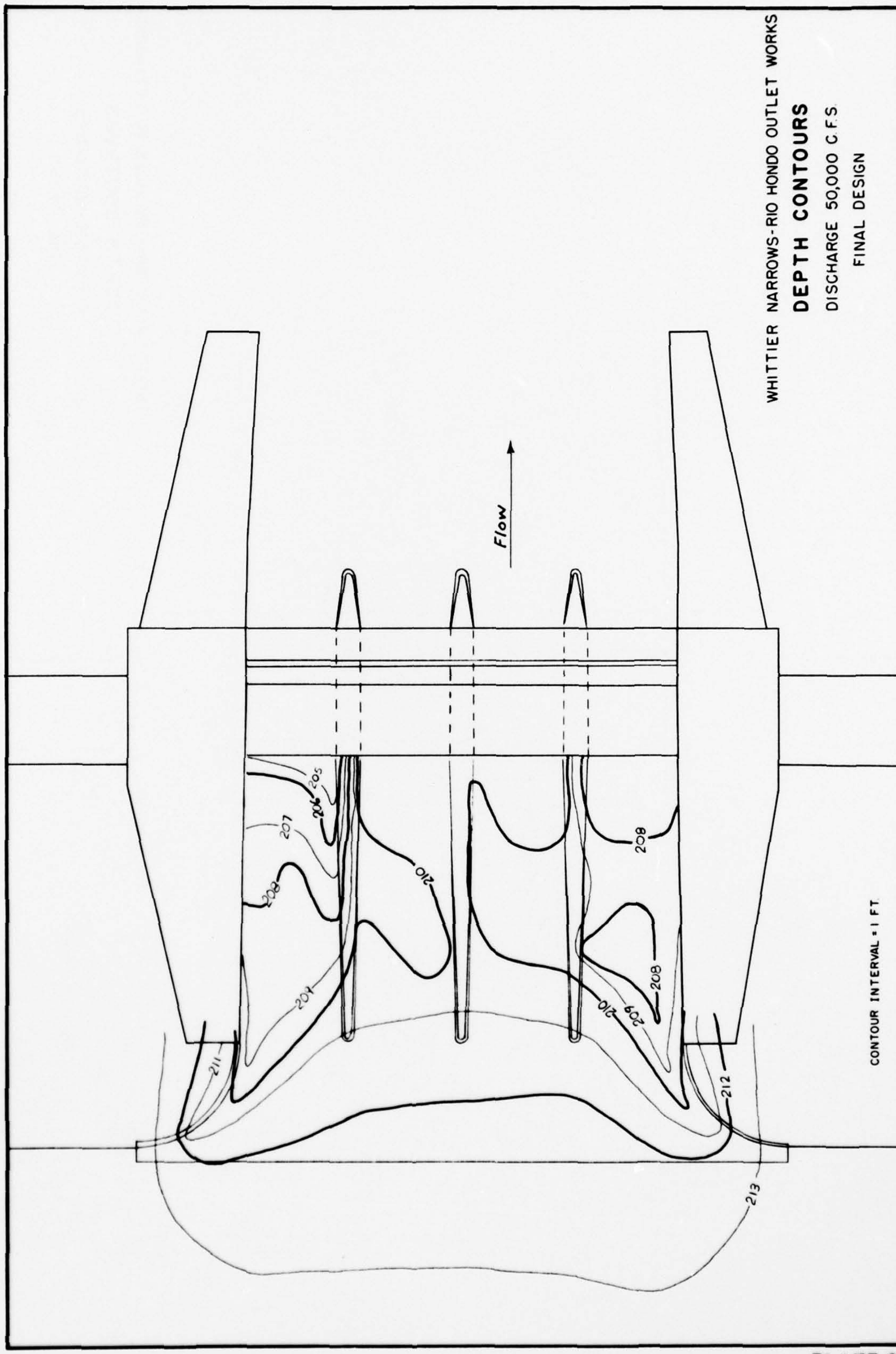


WHITTIER NARROWS– RIO HONDO OUTLET AND CHANNEL

WATER-SURFACE PROFILES

DISCHARGE: 40,000 CFS; GATES OPEN

FINAL DESIGN


WHITTIER NARROWS-RIO HONDO OUTLET WORKS
DEPTH CONTOURS
DISCHARGE 50,000 C.F.S.
FINAL DESIGN
Flow
CONTOUR INTERVAL = 1 FT.
205
206
207
208
209
210
211
212
213

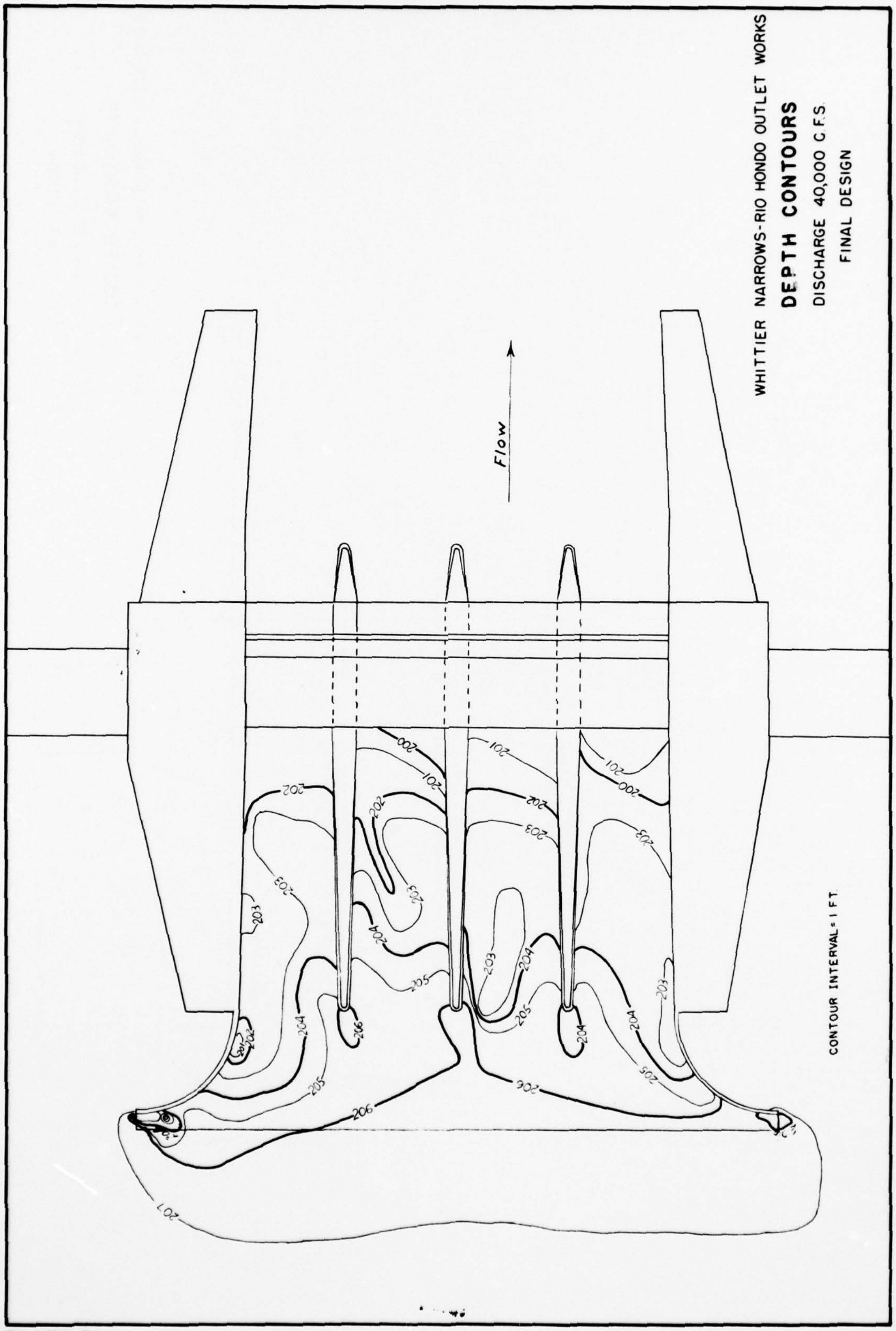

WHITTIER NARROWS-RIO HONDO OUTLET WORKS
DEPTH CONTOURS
DISCHARGE 40,000 C.F.S.
FINAL DESIGN
Flow
CONTOUR INTERVAL = 1 FT.

PLATE 26

PART IV: GENERAL MODEL OF WHITTIER NARROWS DAM

Technical Features of Model Construction

Required conditions

41. In establishing the hydraulic merits of the spillway structure design features as to their adequacy in a unified performance and in assuring protection for the stability of the structure, the following effective factors of the prototype required consideration in determining the construction and behavior of the model:

- a. The best type of training walls for maintaining a smooth flow condition through the gate section.
- b. The most satisfactory type and size of dentates for even distribution of flow at the end sill.
- c. The type and depth of derrick stone protection.
- d. The flow conditions in the channel downstream from the spillway structure.
- e. The probable effect of scour depth downstream during flood flows on the efficiency of the cutoff wall, dentates, and derrick stone aprons.

Description of model

42. The comprehensive model, constructed to a scale ratio of 1:60, model to prototype, reproduced the San Gabriel River spillway structure, the Rio Hondo outlet structure, the Rosemead Blvd. Bridge and drop structure, the flood-flow channel, and the San Jose Creek Diversion-San Gabriel River confluence. The reservoir area, although not reproduced in its entirety, was large enough to produce the characteristic flow required in the operation of the pertinent features of the model. The embankment forming the dam, the levees forming the outlet and spillway channels, and the topographic features of the basin were molded in cement mortar except the San Gabriel River inflow channel. The portion of the model representing the spillway, outlet structures, and piers was constructed of waterproofed wood to simulate reinforced concrete. Crushed rock was used to simulate the derrick stone which was placed at the downstream end of the spillway structure to minimize scour. The

tainter gates were fabricated of wood and sheet metal. The spillway gates were operated by a reduction gear box on each end of the gate assembly. The streambed, molded in sand, was instrumental in the determination of the scour. The Rosemead Blvd. drop structure was molded in concrete. The bridge is comprised of four wooden piers. The model configuration of the reservoir area is shown in Photos 22 and 23. Photo 22 also illustrates the curves in dam alignment near the spillway and outlet structure.

Scale relations

43. General relations for the transference of model data to prototype equivalents or vice versa are presented in the following tabulation:

Dimension	Ratio	Scale Relations
Length	L_r	1:60
Area	$A_r = L_r^2$	1:3,600
Velocity	$V_r = L_r^{1/2}$	1:7.75
Time	$T_r = L_r^{1/2}$	1:7.75
Discharge	$Q_r = L_r^{5/2}$	1:27,885
Volume	$V_r = L_r^3$	1:216,000

Procedure and Accomplishment of General Model Tests

Spillway structure

44. Procedure of tests. The general procedure in testing the designs was: (a) study of the currents in the reservoir to determine their influence on the stability of the structure and operation of the spillway; (b) study of the approach conditions to determine the design of the upstream wing walls and the location of the automatic control system intakes; (c) study of the flow conditions to determine the operating characteristics of the spillway gates and the clearance at the gate counterweights; (d) study of the flow emitting from the spillway to

determine the type, shape, and size of end sill; (e) measurements of water-surface elevations to determine the relations of the reservoir pool elevation and the water-surface elevations at the automatic control and recorder well intakes, and (f) measurements of the scour patterns downstream of the spillway to determine the shape of the downstream wing walls, the depth of cutoff wall, and the extent of rock protection required downstream of the end sill and along the toe of the levees. The model was operated as the prototype would be in case of a spillway design flood. Operation was based on a hydrograph. Flow patterns through the reservoir during the start of the hydrograph are shown in Photo 23. The maximum reservoir water-surface elevation for the spillway design flood would be 234.0. The corresponding maximum outflow would be 40,000 cfs in the Rio Hondo channel and 250,000 cfs into San Gabriel River, a total discharge of 290,000 cfs.

45. Description of tests. Tests on the operation of the spillway were made on the general model with a discharge of 252,000 cfs to determine depths and velocities in the approach to the spillway, in the gate bays, and in the channel downstream from the gates. The maximum velocity in the approach to the spillway was 26 fps in the drawdown near the right wing wall. The magnitude and direction of approach currents are given in Plate 27. Observations made during preliminary tests for a discharge of 252,000 cfs (spillway design discharge is 254,000 cfs) indicated that the water surface should be lower in the vicinity of the gate counterweights. It was found that by offsetting the walls of the spillway channel immediately downstream from the gates by 4 ft (equivalent to half a pier width) greater clearance between the water surface and the counterweights could be obtained. The wider downstream channel was necessary to eliminate the standing wave which would impinge against the gate counterweights at the right and left walls. Photographs of the model with gates discharging into a 530-ft-wide channel are shown in Photo 24. The upper right photograph shows the standing wave at the left wall where the gate counterweight would be; increasing the width to 538 ft eliminated this wave.

46. Tests were made of the 35-ft radius wing walls, used in the

contract plans, and 60.5-ft-radius wing walls. Photo 24 shows the models in operation. Profiles of the water surface for a discharge of 250,000 cfs for both wing walls are shown in Plate 28. The maximum drawdown for the 35-ft radius wing wall was 20.4 ft and for the 60.5-ft-radius wing wall was 17 ft. The reduction in drawdown that would result from the use of a 60.5-ft-radius wing wall was not considered sufficient to warrant redesign of the wing-wall structure. Water-surface elevations (cross sections) are given in Tables 5, 6, and 7. The concrete apron as shown in Plate 4 would extend 25 ft from the ends of the wing walls to resist the scouring effects created by the drawdown around the ends of the wing walls.

47. Spillway design 1 - basic design. In the original design, the spillway had a constant width of 530 ft. Above and below the gates, the walls were straight and parallel. The radii of the quadrant wing walls at the upstream and downstream ends of the spillway structure were 35 ft and 75 ft, respectively. The dentates with sloping upstream faces, 8 ft high, 5 ft wide, and spaced 5 ft apart, were placed at the end sill. For the test, various inflows indicated by the spillway design-flood hydrograph were introduced from the Rio Hondo through the flood-flow channel and from the San Gabriel River. Table 8 gives ranges of discharges, corresponding pool elevations, and gate openings. The total time of the hydrograph run was 42 hr (prototype). The spillway model is shown in Photo 25; the hydrograph run is shown in Photo 26. Two scour holes formed off the end of the structure, one at each side of the channel. The abrupt change in cross-sectional area of flow passing from the spillway channel to the downstream leveed channel contributed much toward the creation of large eddies over the area where the scour holes formed. Scour photographs are shown in Photo 27; the scour pattern is shown in Plate 29.

48. Revision of basic design. The first revision of the spillway structure to reduce the depth of flow immediately downstream of the gates consisted of widening the spillway channel downstream of the gate piers from a width of 530 ft to 538 ft. In addition to the rock protection below the end sill, rock protection was added along the toe of both

levees (see paragraph 54 for description of the rock); this change is designated spillway design 2. Photo 28 shows the rising stage of the hydrograph run. Observations of flow through the spillway structure over the full range of discharges showed that the performance of this spillway design was satisfactory. No disturbance in flow was noted. For flows up to 254,000 cfs (spillway design discharge), the gate counterweights were clear of flow. Photo 29 shows results of scour; the scour pattern is shown in Plate 30. Comparison of the results of this design with those of spillway design 1 showed that less scour occurred in the center of the channel below the end sill, but the scour holes at the sides of the channel near the quadrant walls were about the same as those in the previous test. Consequently, all efforts were directed toward developing a suitable type of dentate that would diffuse the flow at the end sill near the quadrant walls and prevent excessive scour at the sides of the channel. Changes were limited to the dentates near the quadrant walls. The following types were tested in deriving a satisfactory end sill.

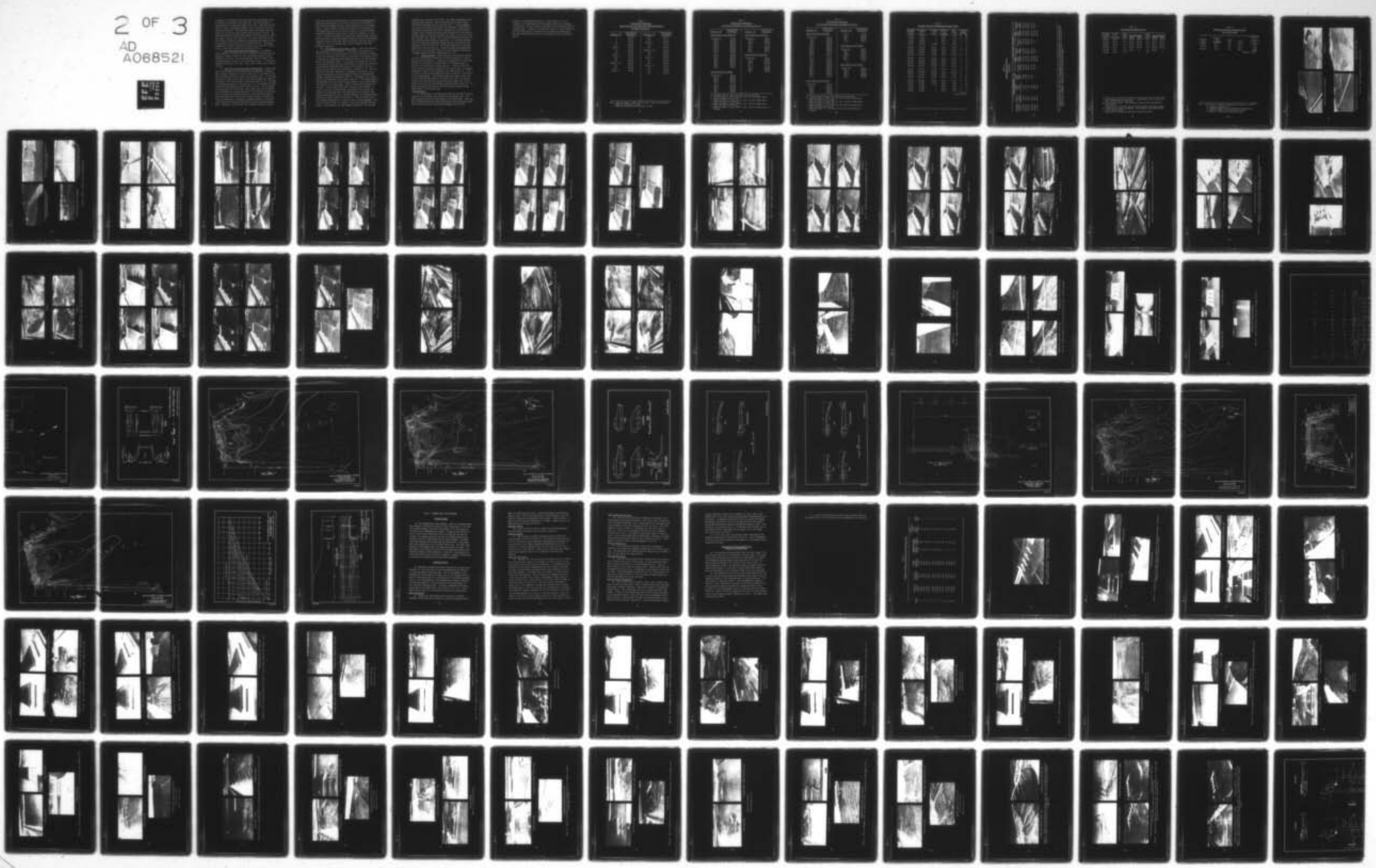
49. Test with larger dentates. Four large dentates having a uniform width of 5 ft were substituted for six original dentates (5 ft by 8 ft). Two were 10 ft high and the other two, 12 ft high (Photo 30, top). Tests showed that very little improvement resulted with use of the larger dentates. Evident in the discharge photograph is the eddy over the area where the scour hole formed. No attempt was made to obtain scour data after the test run.

50. Test with 30- and 36-ft blocks. The four large dentates used in the previous test were replaced with two 8-ft vertical blocks, 30 and 36 ft long (for details see Plate 31). The blocks were 13 ft apart and in staggered alignment with the row of dentates. The longitudinal center line of each block was skewed to the left 12.5 deg with the longitudinal center line of the spillway channel, as shown in Photo 30. However, no beneficial effect was indicated with this plan. The general flow pattern was about the same as in previous tests. Eddies still persisted at the side of the channel. No scour measurements were taken.

51. Test with curved guide vanes (design 1). Dentates 1, 3, and

AD-A068 521 ARMY ENGINEER DISTRICT LOS ANGELES CALIF F/G 13/2
WHITTIER NARROWS FLOOD-CONTROL BASIN, LOS ANGELES COUNTY DRAINA--ETC(U)
FEB 79 D A BARELA
UNCLASSIFIED 2-112 NL

2 OF 3
AD
A068521



5 adjacent to the quadrant walls were skewed and each provided with a 5-ft-high by 2.5-ft-wide curved guide vane. The only variation in design involved the radius and length of the vane. Radii were 90, 100, and 150 ft with corresponding lengths of 60, 54, and 48 ft (for vane details see Plate 32). The guide vane nearest the quadrant wall had a radius of 90 ft and the vane farthest away had radius of 150 ft. The added vanes showed a definite improvement in the distribution of flow at the sides of the end sill (Photo 31). The curved vanes directed the flow outward, reducing the eddies and consequently minimizing the scour at each side and off the end of the spillway structure. Testing was accomplished by visual observation and results were recorded by photographs. Scour after a 30-min run is shown in Photo 32.

52. Test with curved guide vanes (design 2). In obtaining a greater degree of diffusing the flow, a modified curved guide vane was prepared, as shown in Plate 33. Such modification produced further spreading of the flow. This vane design was found to produce a more favorable hydraulic action and less scour than design 1. No scour measurements were made on this test. Scour after a 30-min run is shown in Photo 32.

53. Tests with curved guide vanes (final design). Although eddy action was decreased and flow distribution was improved, it appeared that a change in design for the guide vanes would be necessary to eliminate the eddy action that existed in the previous tests. In this test the number of vanes at each side of the spillway channel was increased from three to four and the width of vane was reduced to 2 ft. The width of the dentates attached to the vanes was 4 ft. The dentates and vanes conformed to the design and layout shown in Plate 34. The development of the dentated sill with the four curved guide vanes and the derrick stone protection at the downstream edge of the dentated sill and along the toe of levees was based on analysis of flow pattern and depth of scour resulting from a 30-min run of the peak outflow of 250,000 cfs. This was accomplished by simulating a maximum discharge of 83,300-cfs inflow from the Rio Hondo and 166,700-cfs inflow from the San Gabriel River. Desirable flow conditions were obtained by using the slender

guide vanes with the narrower dentates. These guide vanes appeared to improve the distribution of flow over the end sill and eliminate the eddy action that formed along the levees downstream from the quadrant walls. A test was made to determine the effect on scouring action in the channel below the end sill and the stability of the derrick stone protection downstream from the end sill under low tailwater. In view of this, a tailwater elevation 10 ft below the computed normal value was used in the test. With this tailwater condition, there was considerable movement of derrick stone away from the spillway cutoff wall and an increase of 12 ft in the maximum depth of scour compared with normal tailwater conditions.

54. Rock protection tests (types 1 and 2). Tests were conducted upon two types of rock protection. In the type 1 rock protection a 10-ft-thick layer of rock was placed on a 1V-on-2H slope for a distance of 60 ft at the middle section and 1V-on-3H slope for a distance of 90 ft at each end of the spillway with transition zones to the rock-protected toes of the downstream levees. In the type 2 rock protection, the 1V-on-2H slope was placed along the entire width of spillway and 1V-on-2.2H slope extending downstream along the side of the levees. The toe of both levees had derrick stone protection to a depth of 12 ft. The rock protection tests were conducted at a discharge of 250,000 cfs, and a check on the adequacy of the rock designs was made by simulating the spillway design-flood hydrograph (Table 8). Photographs of the rising stage of the hydrograph run are shown in Photo 33. The falling stage of the hydrograph run is not shown but was executed in the reverse order of that shown for the rising stage. Photo 34 shows scour photographs for the type 1 rock protection and Plate 35 gives the scour pattern. In tests on the type 2 rock protection, the operation of the model was stopped on the receding side of the hydrograph run (240,000 cfs) and scour measurements were taken. The scour pattern for the partial hydrograph run is shown in Plate 36 and Photo 35. The scour pattern after the completed hydrograph run is shown in Plate 37 and photographs of the scour are shown in Photo 36. Results of the tests indicated that both types of rock protection provided adequate protection to the spillway

structure and to the toe of the levees. No change was apparent in the resulting scour patterns. The scour patterns in the general model verified those obtained in the 1:42-scale model. It was recommended that type 1 rock protection be used in the prototype. Measurements of water-surface elevations were made to determine the relation between reservoir pool elevation and water-surface elevations at right and left automatic control intakes and recorder well intakes for both free flow and controlled flow. Tables 9 and 10 show water-surface elevations and gate operating characteristics, respectively. The spillway discharge curves for controlled discharges through the gates with partial gate openings are shown in Plate 38. The average coefficient of discharge computed for free flow was 2.81. Several discharge measurements were made in the 1:42-scale model (Table 11) and the average coefficient of discharge computed from these measurements was 2.80.

55. Single bay model. Having developed a satisfactory spillway with its rock protection downstream, the next problem was to evolve a suitable upstream approach. A qualitative determination of scour depth upstream of the concrete apron was made by a 1:24-scale model of a section of the approach to a single gate bay. A test was first conducted without a rock apron upstream of the concrete slab. A prototype discharge of 29,000 cfs was simulated in the model (Photo 37). Scour profiles for various time durations are shown in Plate 39. See Photo 38 for scour photographs. Two rock apron plans were tested in the model. In both plans, the apron was 25 ft wide and 3 ft thick. One plan was normal to the flow line and the other was skewed (Photo 39). Photographs of the scour for the rock apron plans are shown in Photo 40. The steel sheet pile cutoff and the rock apron provided should preclude undercutting of the concrete slab.

Outlet structure

56. Verification of performance and tests of the outlet structure tested in the 1:24-scale model were made in the general model. The model design simulated the final design of the outlet works. Tests were made to determine the water depth differential along the quadrant walls of the outlet works with uncontrolled discharges of 40,000 and

50,000 cfs. The maximum differential in water depth of 6 ft was observed at the downstream end of the quadrant walls with a discharge of 40,000 cfs. For the 50,000-cfs discharge, the differential depth was slightly larger. Results of the tests were similar to those obtained in the 1:24-scale model and indicated that the design was satisfactory. The flow characteristics for the two discharges are shown in Photos 41 and 42.

Table 5

Water-Surface Elevations

Cross Section at Sta 9+53, 35-ft-Radius Wing Walls;

Discharge 252,000 cfs

Distance, ft*	Water-Surface Elevation	Distance, ft*	Water-Surface Elevation
0	224.30	260	230.54
6	225.50	280	230.12
18	223.40	℄ pier 6	234.80
33	226.64	318	230.60
℄ pier 1	234.14	334	230.00
78	229.52	℄ piers 7 & 8	234.80
98	230.06	389	230.48
℄ pier 2	234.32	406	229.94
136	230.78	℄ pier 9	234.56
156	230.60	444	229.18
℄ piers 3 & 4	234.86	426	229.16
207	230.96	℄ pier 10	234.32
227	230.60	500	225.32
℄ pier 5	234.86	520	219.80
		530	219.20

Note: Reservoir gages: right = 234.54, left = 234.40; intake wells: right = 233.70, left = 234.08

* Distances measured from left wall, in feet.

Table 6

Water-Surface Elevations

35-ft-Radius Wing Walls; Discharge 252,000 cfs

Distance, ft*	Water-Surface Elevation
Cross Section at Sta 8+92.5**	
Center line	231.38
100 R	231.02
200 R	228.79
231 R	225.74
260 R	219.56
294 R	215.36
300 R	232.04
350 R	233.00
400 R	233.24
500 R	233.26
100 L	231.56
200 L	230.42
247 L	228.14
268 L	225.50
285 L	221.54
300 L	233.24
350 L	233.78
--	--
Cross Section at Sta 8+60†	
Center line	231.68
100 R	231.32
200 R	229.58
267 R	228.56
300 R	231.38
350 R	233.00
500 R	233.00
100 L	231.68
200 L	231.32
350 L	233.60
Cross Section at Sta 7+50††	
Center line	232.88
50 R	232.82
200 R	232.70
350 R	233.42
500 R	233.48
50 L	233.94
200 L	233.42
350 L	233.72
Cross Section at Sta 6+50‡	
Center line	233.60
50 R	233.44
200 R	233.48
350 R	233.66
500 R	233.66
50 L	233.60
200 L	233.84
350 L	233.66

* Distance left and right from center line of spillway.

** Reservoir gages: right = 234.30, left = 234.16; intake wells: right = 233.52, left = 233.90.

† Reservoir gages: right = 234.04, left = 234.16; intake wells: right = 233.70, left = 234.02.

†† Reservoir gages: right = 234.23, left = 234.22; intake wells: right = 233.58, left = 234.02.

‡ Reservoir gages: right = 234.24, left = 234.16; intake wells, no readings.

Table 7

Water-Surface Elevations

60.5-ft-Radius Wing Walls; Discharge 252,000 cfs

Distance, ft*	Water-Surface Elevation
Cross Section at Sta 9+33**	
Center line	230.71
27 R	231.07
60 R	230.89
91 R	231.35
124 R	230.35
153 R	230.35
182 R	228.67
207 R	228.37
236 R	222.85
265 R	217.33
300 R	233.17
400 L	233.35
500 R	233.05
28 L	230.95
58 L	230.95
91 L	231.67
125 L	230.89
155 L	230.77
185 L	229.99
210 L	229.21
245 L	223.99
265 L	221.45
230 L	233.65
--	--
Cross Section at Sta 8+92.5†	
Center line	231.55
100 R	231.07
150 R	230.59
235 R	228.73
275 R	227.11
340 R	232.99
400 R	233.23
100 L	231.67
275 L	229.63
300 L	222.99
371 L	233.51
--	--
Cross Section at Sta 8+60††	
Center line	231.85
200 R	230.47
300 R	229.27
400 R	233.11
500 R	233.11
200 L	231.73
350 L	233.23
--	--
Cross Section at Sta 6+50‡	
Center line	233.65
200 R	233.59
400 R	233.77
500 R	233.65
200 L	233.89
350 L	233.89

* Distance left and right from center line of spillway.

** Reservoir gages: right = 233.95, left = 234.04; intake well: right = 233.04, left = 233.90.

† Reservoir gages: right = 234.12, left = 234.28; intake wells: right = 233.10, left = 233.96.

†† Reservoir gages: right = 234.06, left = 234.22; intake wells: right = 233.46, left = 233.90

‡ Reservoir gages: R = 234.00, left = 234.22; intake well, no readings.

Table 8

Spillway Design-Flood Hydrograph, General Model

Spillway Discharge cfs	San Gabriel Inflow cfs	Rio Hondo Inflow cfs	Reservoir Pool Elevation	Gate Opening ft	Prototype Time hr	min
40,000	40,000	0	230.0	2.85	0	30
70,000	70,000	0	230.6	5.20	0	30
90,000	90,000	0	230.9	6.95	0	30
110,000	100,000	10,000	231.2	8.80	5	15
130,000	100,000	30,000	231.5	10.85	8	30
150,000	100,000	50,000	231.8	13.05	2	30
170,000	113,600	56,600	232.2	15.70	3	15
200,000	133,000	66,700	232.7	19.70	0	45
220,000	146,700	73,300	233.0	21.70	0	30
240,000	160,000	80,000	233.3	23.30	1	0
250,000	166,700	83,300	233.9	25.00	2	45
240,000	160,000	80,000	233.3	23.30	0	45
220,000	146,700	73,300	233.0	21.70	0	45
200,000	133,300	66,700	232.7	19.70	1	0
170,000	113,400	56,600	232.2	15.70	0	45
150,000	100,000	50,000	231.8	13.05	0	45
130,000	100,000	30,000	231.5	10.85	0	45
110,000	100,000	10,000	231.2	8.80	0	45
90,000	90,000	0	230.9	6.95	1	30
70,000	70,000	0	230.6	5.20	4	0
40,000	40,000	0	230.0	2.85	5	0
					Total = 42 hr	

Table 9

Water-Surface Elevations

Free Flow

	Initial Conditions			Water-Surface Elevations					
				Right			Left		
Prototype Discharge cfs	Reservoir Pool Elevation	San Grabriel Inflow Model cfs	Rio Hondo Inflow Model cfs	Reservoir Pool	Auto. Control Intake	Recorder Well Intake	Reservoir Pool	Auto. Control Intake	Recorder Well Intake
300,000	238.3	7.172	3.586	238.28	237.32	237.86	238.36	238.04	238.10
250,000	233.9	5.977	2.988	233.84	233.06	233.18	233.86	233.54	233.60
200,000	229.2	4.781	2.391	229.22	228.38	228.74	229.30	228.80	228.98
150,000	224.1	3.586	1.793	224.18	222.92	223.40	224.20	223.76	223.76
100,000	218.4	3.586	--	217.88	216.86	217.04	--	217.40	217.34
50,000	210.6	1.793	--	--	210.56	--	--	210.56	--

Note: Established initial conditions. Outlet gates closed. Lifted gates clear of water surface and allowed flow to stabilize. Recorded simultaneously water-surface elevations at right and left reservoir pool, automatic control intakes, and recorder well intakes.

Table 10

Gate Operating Characteristics

Prototype Discharge cfs	Initial Pool Elevation*	Gate Opening ft**	Pool Elevations† Right	Pool Elevations† Left	Gate Opening ft††	Pool Elevations‡ Right	Pool Elevations‡ Left
300,000	238.3	23.2	238.34	238.48	26.2	238.28	238.60
250,000	233.9	20.5	233.84	233.86	24.3	234.32	234.22
200,000	229.2	17.3	229.40	229.36	21.7	229.52	229.72
150,000	224.1	14.3	224.42	224.38	18.1	224.84	224.80
100,000	218.4	10.8	218.24	--	13.2	218.66	--
50,000	210.6	6.5	--	--	8.7	--	--

* Established initial pool elevation. Raised gates clear of water surface. Allowed flow to stabilize.

** Lowered gates slowly. Gate opening at which contact was made with water surface.

† Reservoir pool elevations when gates made contact with water surface.

†† Raised gates by small increments. Gate opening at which gates break free of water surface.

‡ Reservoir pool elevations when gate contact was broken.

Table 11

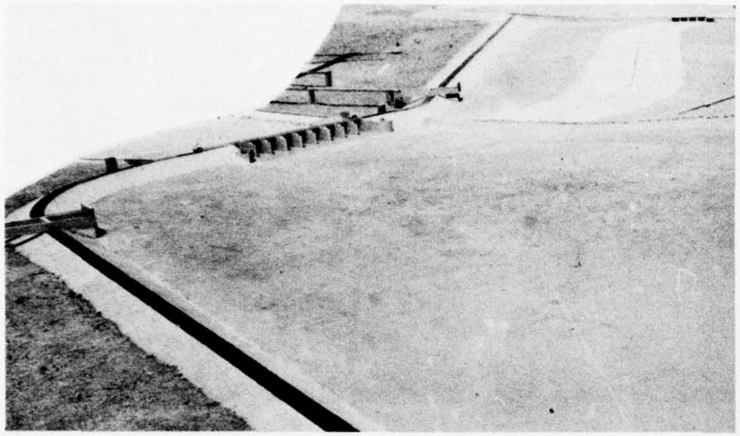
Discharge Coefficient from Model Study

Gates Fully Opened

Discharge cfs	Reservoir Pool Elevation	Head ft	$H^{3/2}$	Coefficient of Discharge C
234,000	232.8	32.8	187.8	2.769
244,000	233.5	33.5	193.9	2.796
252,000	234.0	34.0	198.2	2.825
				Average C = 2.80

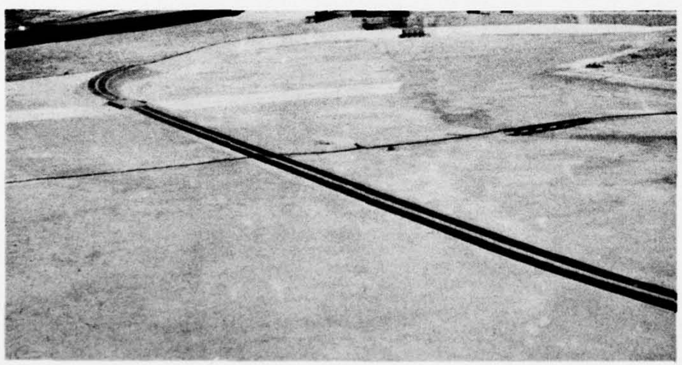
Note: Coefficient of discharge determined from formula $Q = CLH^{3/2}$

Q = Measured discharge, cfs
C = Computed coefficient of discharge from model data
L = Effective length, nine 50-ft gates = 450 ft
H = Head, ft. Gate sill elevation at 200.0.



Spillway side and east embankment

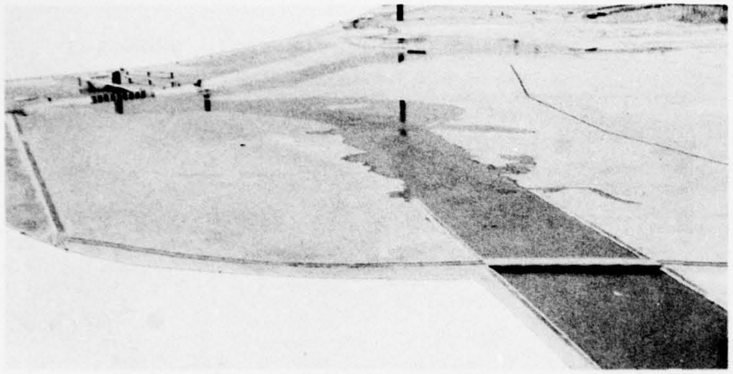
Outlet side



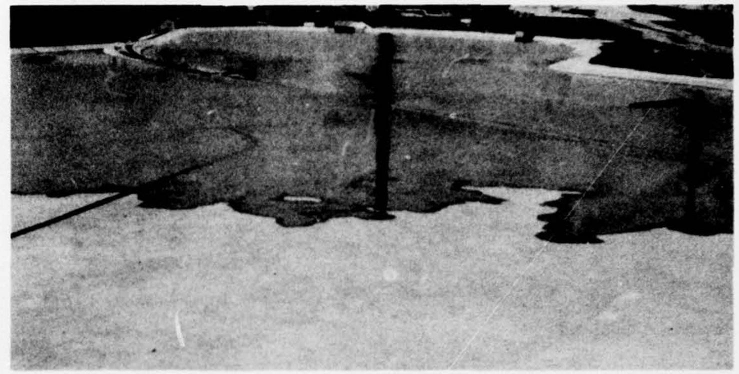
Rosemead Blvd. Bridge and drop structure

San Gabriel River inlet

Photo 22. Model configuration of the reservoir area



Flow from San Gabriel River, looking downstream toward spillway

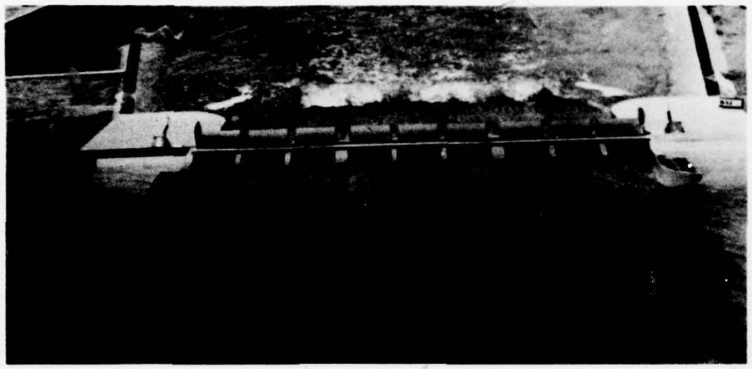


Flow from Rio Hondo, looking downstream toward outlet works

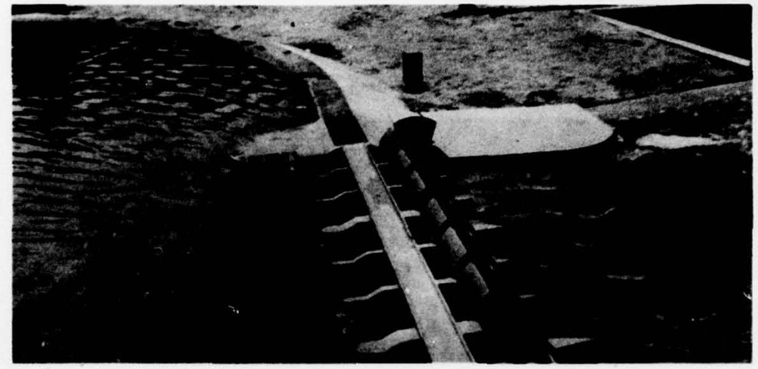
Low flow along central embankment

Flow along central embankment and through the Rosemead Blvd. drop structure

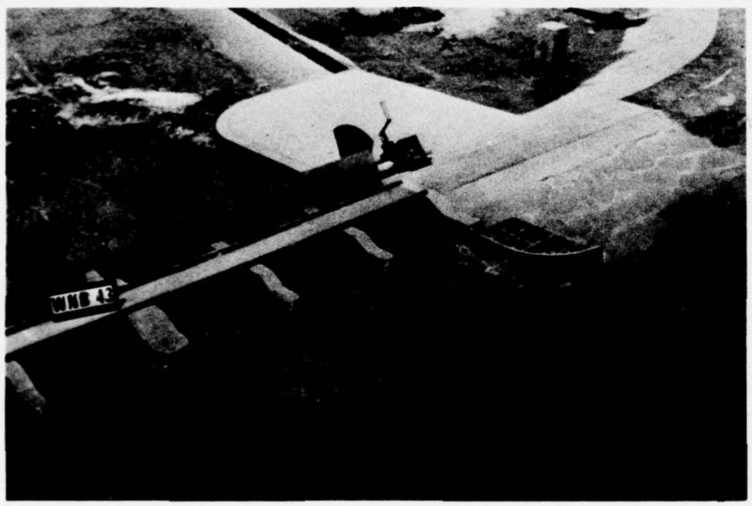
Photo 23. Model in operation, start of hydrograph run



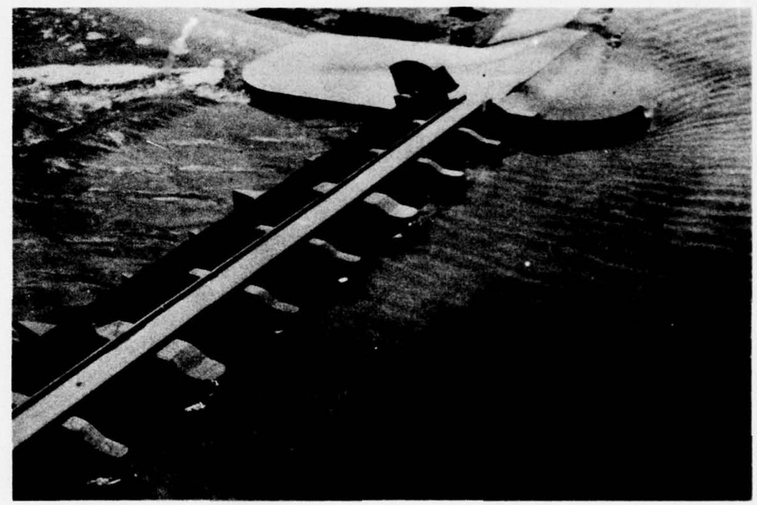
Looking downstream



View from right side

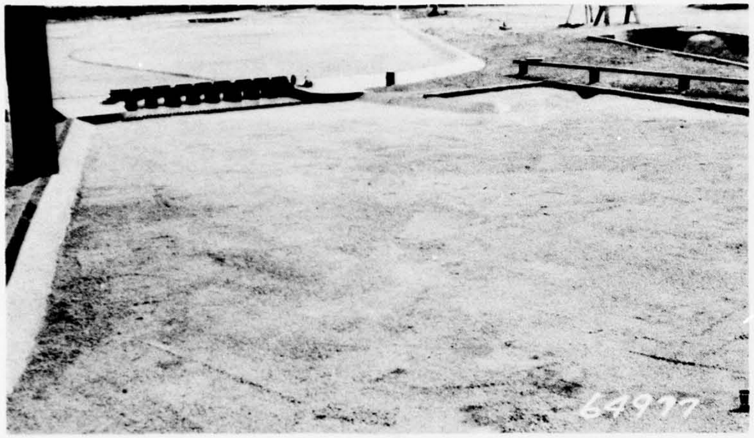


Radius of wing wall 35.0 ft

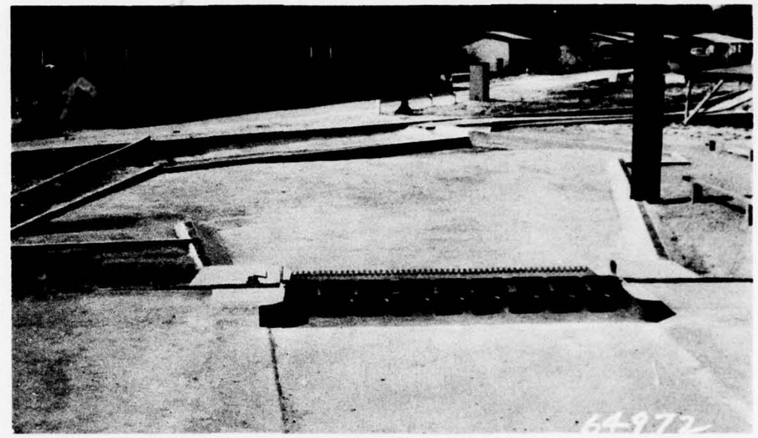


Radius of wing wall 60.5 ft

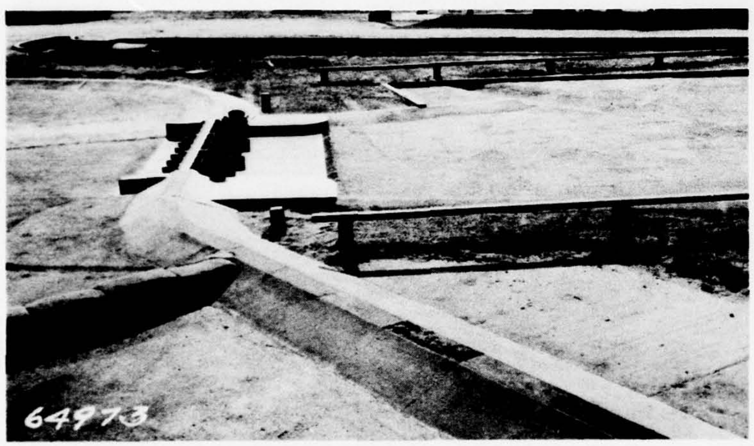
Photo 24. General spillway model, discharge 250,000 cfs. (Note drawdown along right quadrant wing wall of approach channel)



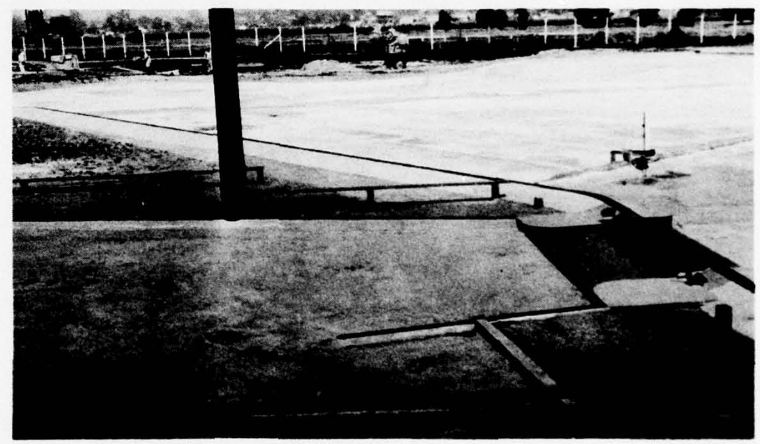
General view, looking upstream



General view, looking downstream

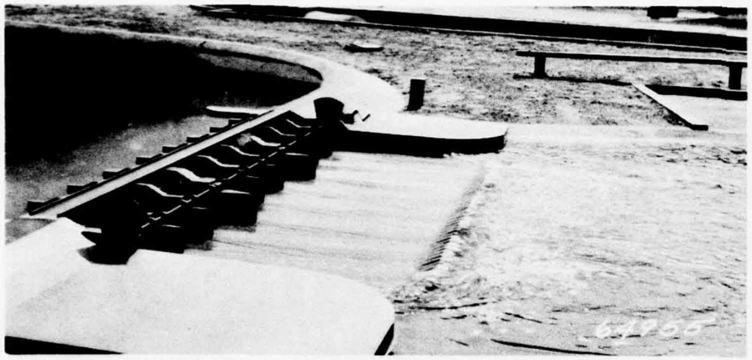


View from right side

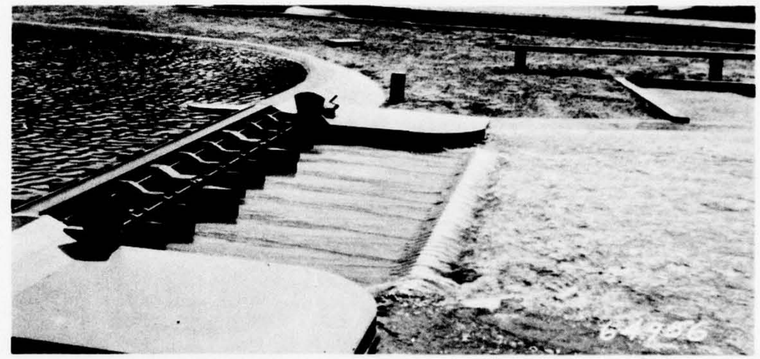


View from left side

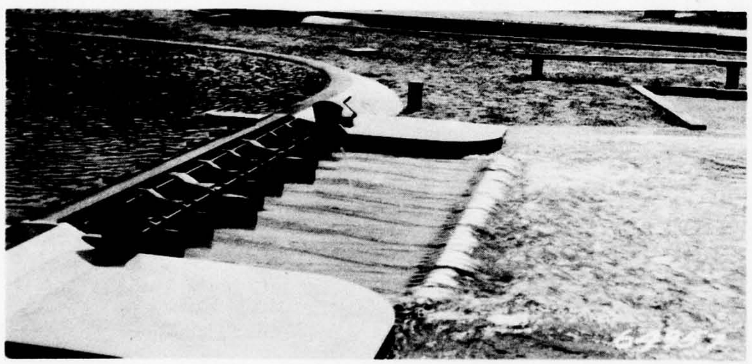
Photo 25. General spillway model



40,000 cfs (rising)



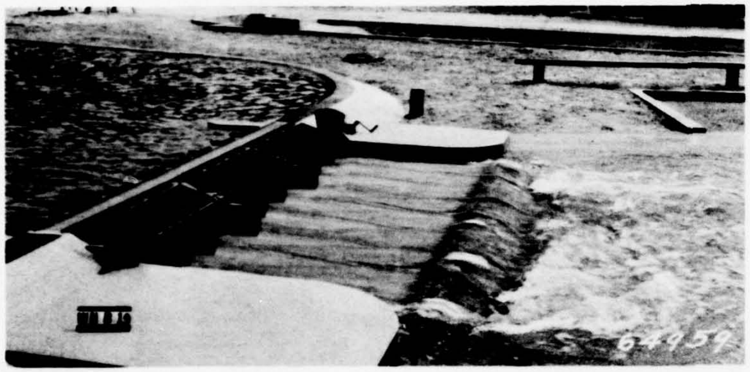
74,000 cfs (rising)



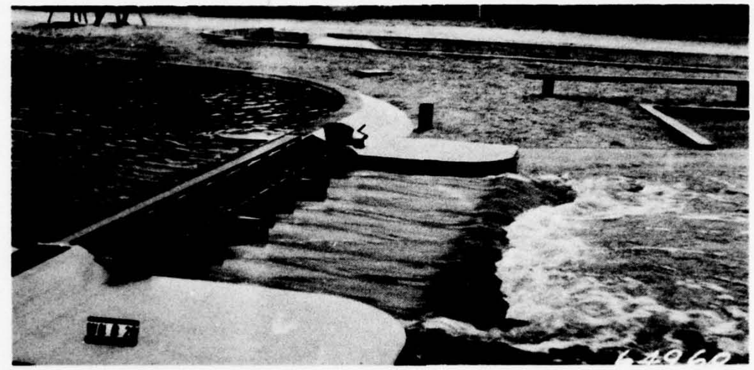
104,000 cfs (rising)

125,000 cfs (rising)

Photo 26. Original design, design-flood hydrograph (sheet 1 of 4)

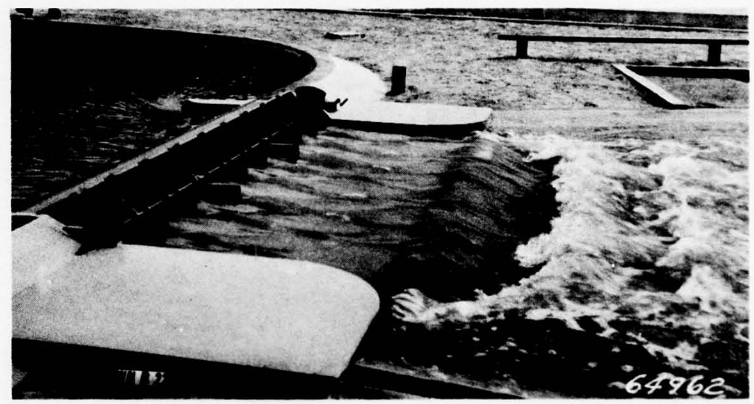


147,600 cfs (rising)



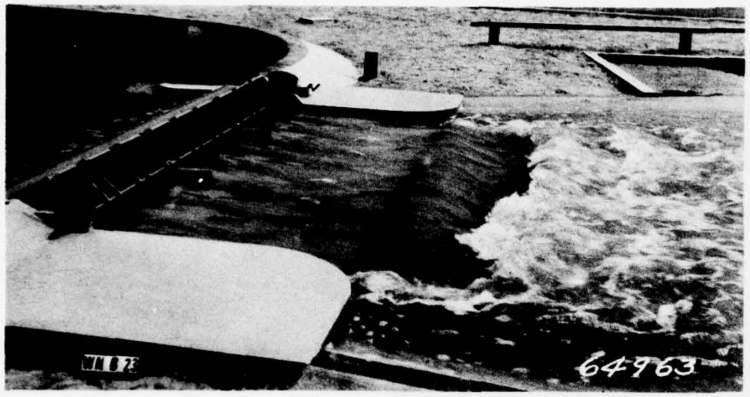
172,400 cfs (rising)

203,000 cfs (rising)

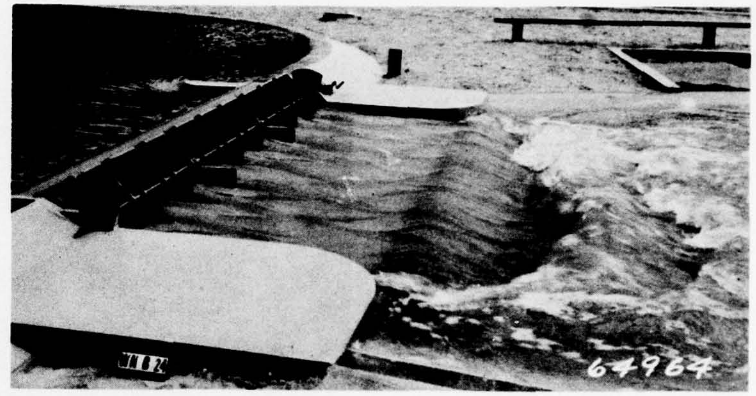


234,000 cfs (rising)

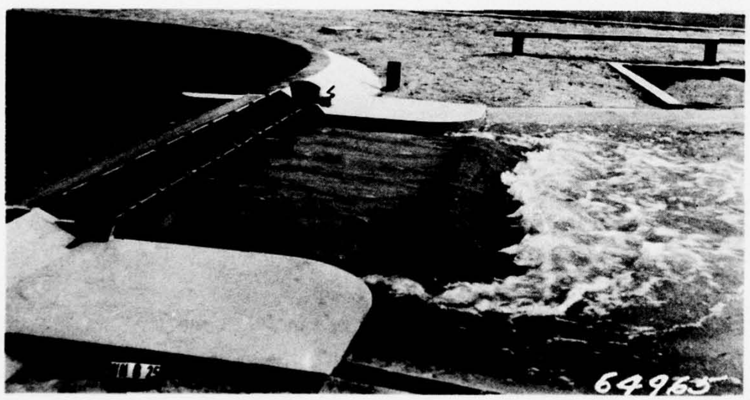
Photo 26 (sheet 2 of 4)



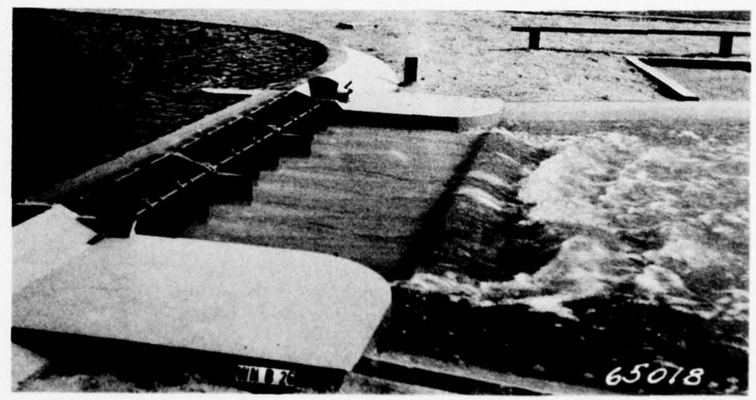
244,000 cfs (rising)



252,000 cfs (peak)

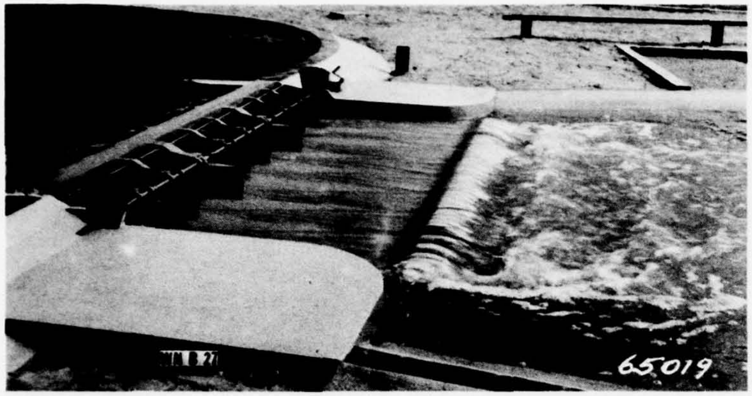


203,000 cfs (falling)

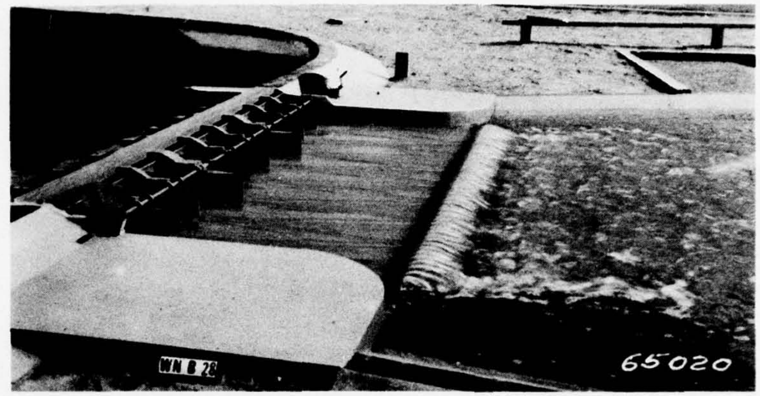


147,600 cfs (falling)

Photo 26 (sheet 3 of 4)



104,000 cfs (falling)



74,000 cfs (falling)

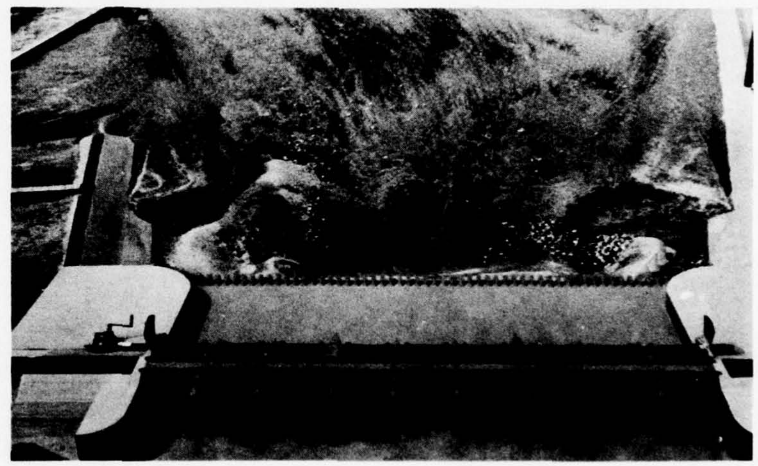
40,000 cfs (falling)

Photo 26 (sheet 4 of 4)

Looking upstream from left side

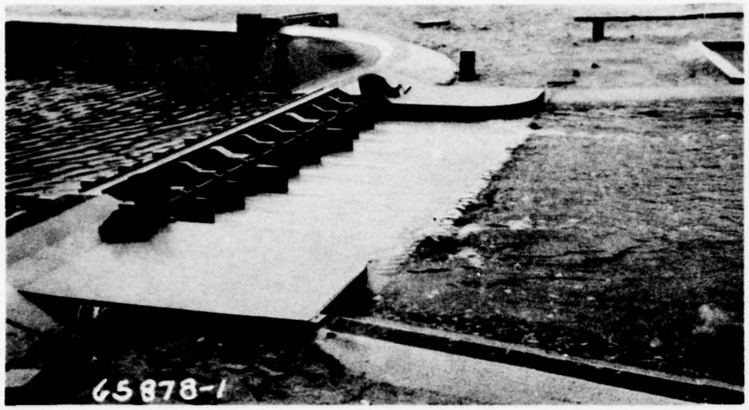
Looking upstream from right side

View across channel from right side (downstream)

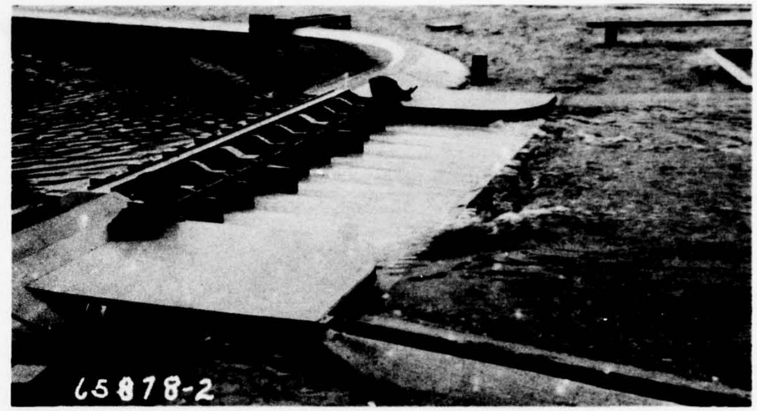


Looking downstream

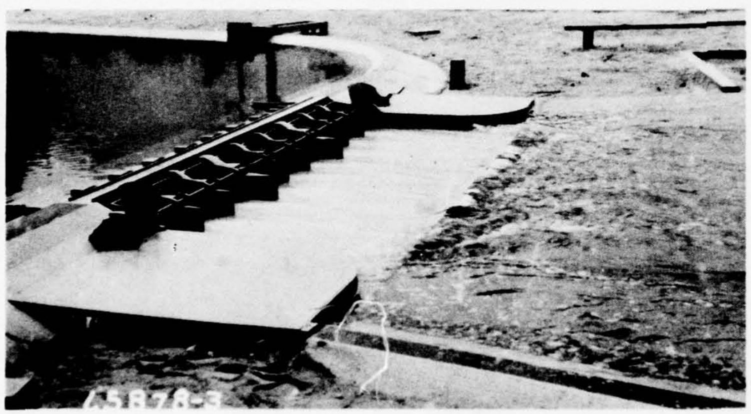
Photo 27. Original design; scour below spillway after hydrograph run



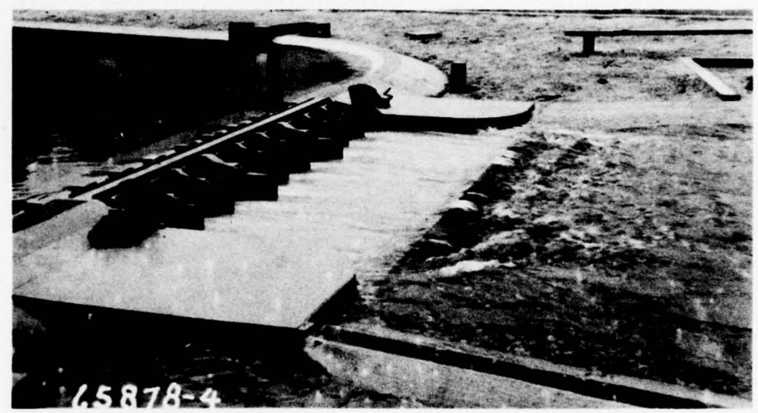
40,000 cfs



70,000 cfs

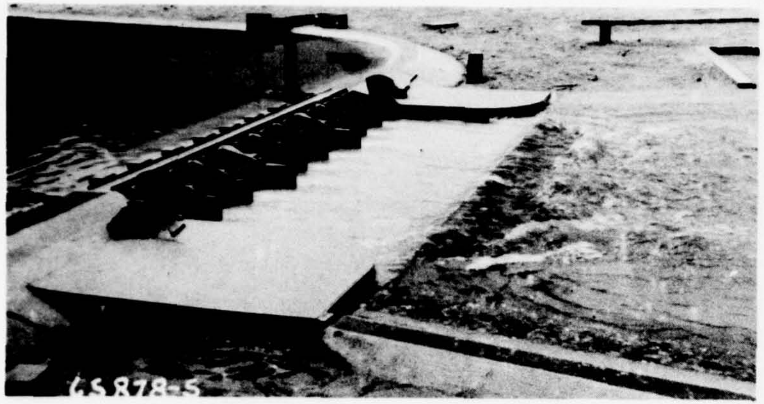


90,000 cfs

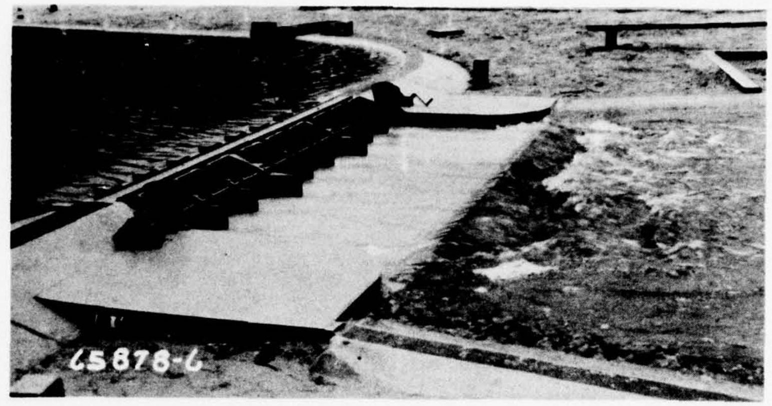


110,000 cfs

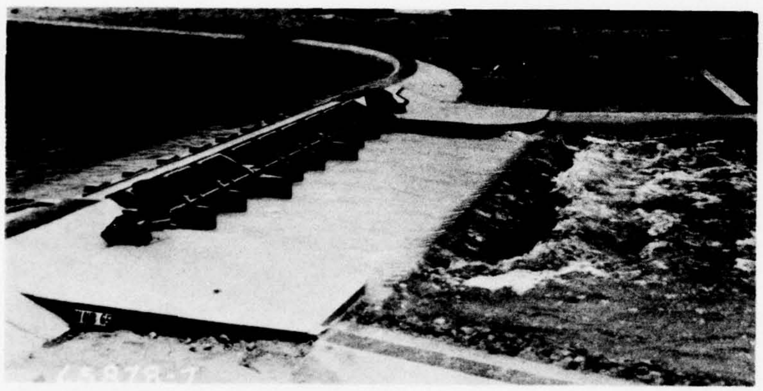
Photo 28. Spillway design 2, design-flood hydrograph (sheet 1 of 3)



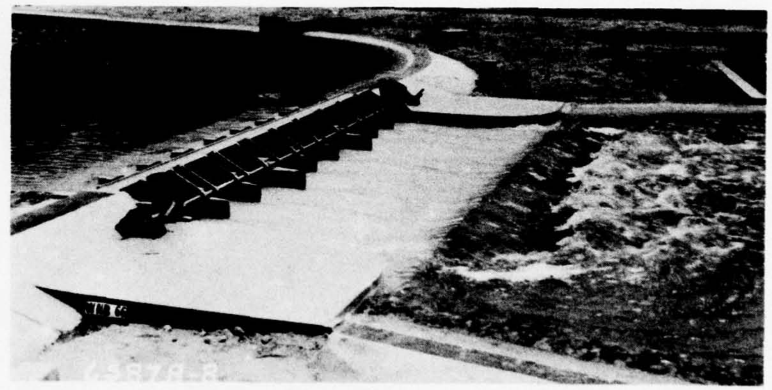
130,000 cfs



150,000 cfs

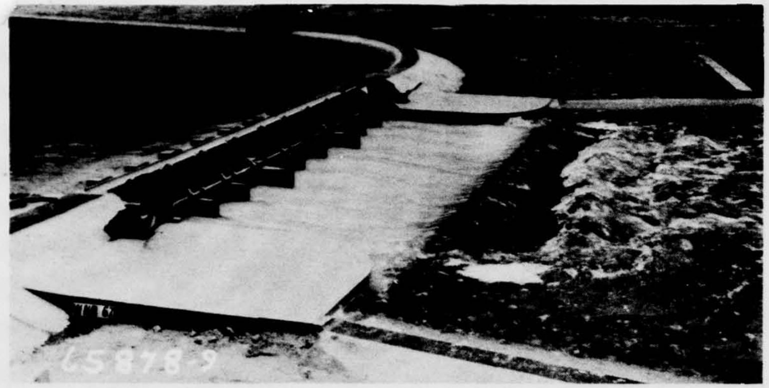


170,000 cfs

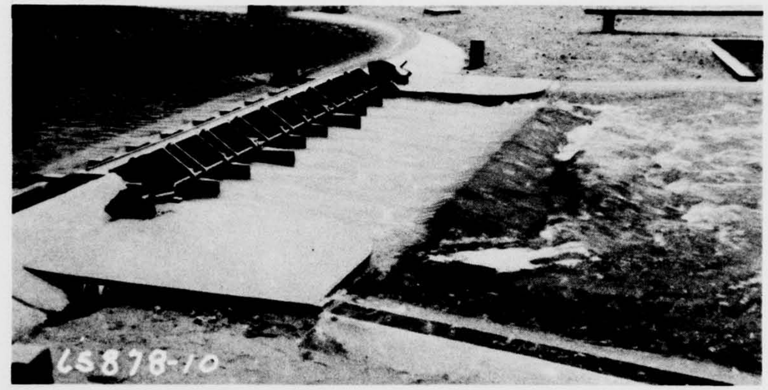


200,000 cfs

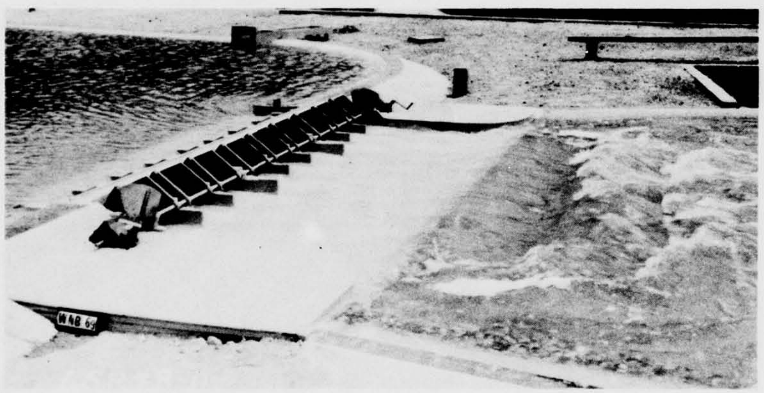
Photo 28 (sheet 2 of 3)



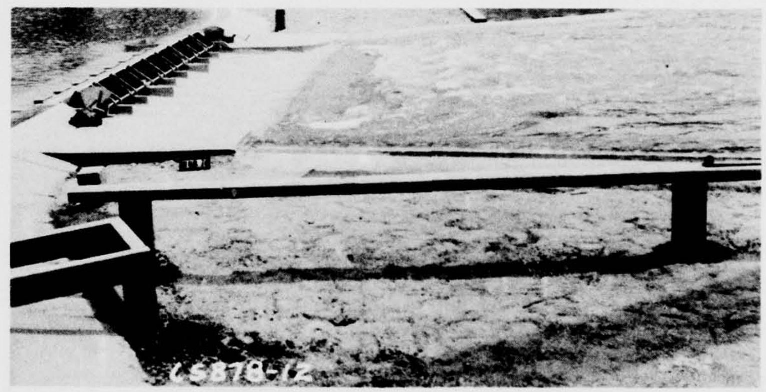
220,000 cfs



240,000 cfs

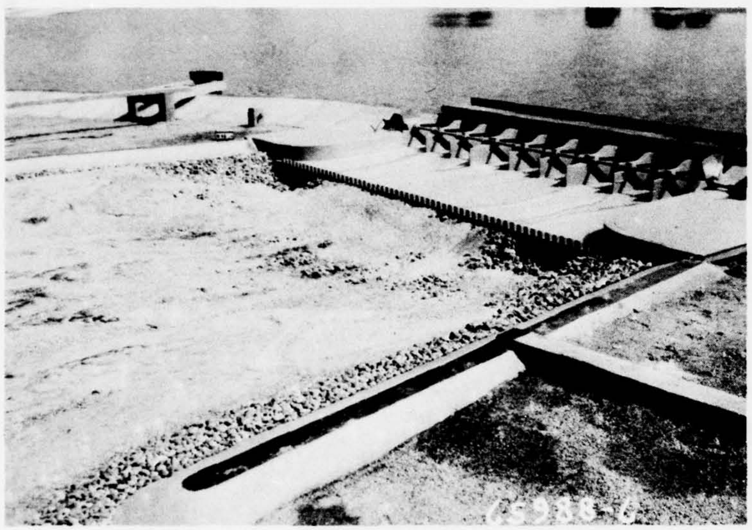


250,000 cfs

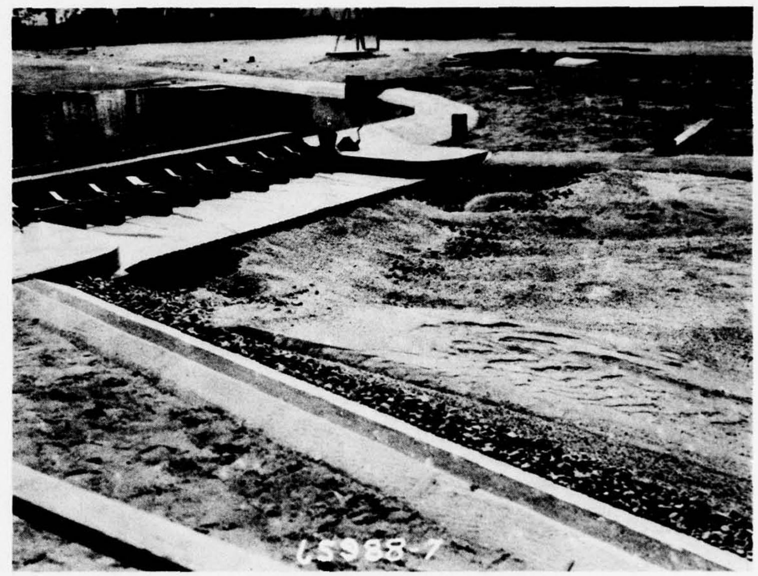


252,000 cfs

Photo 28 (sheet 3 of 3)

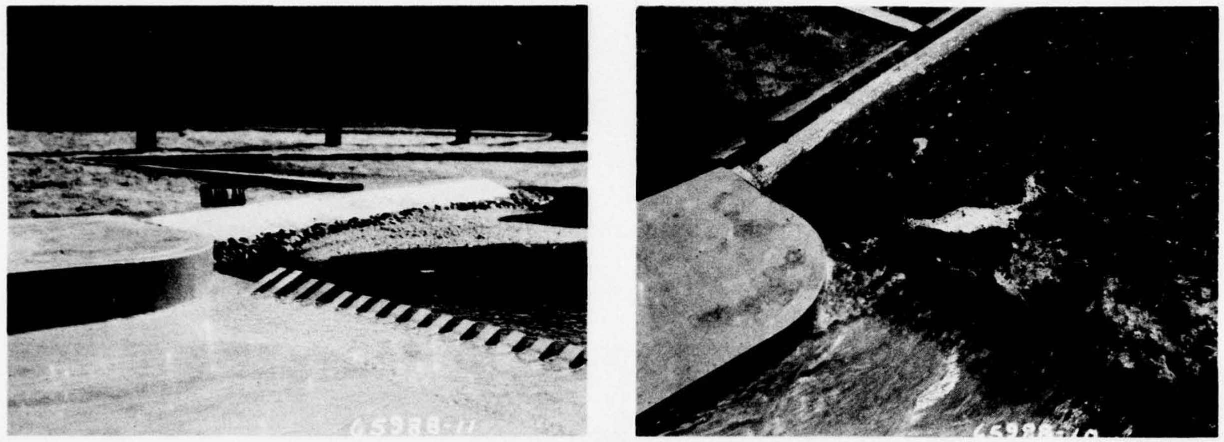


Looking upstream from left side

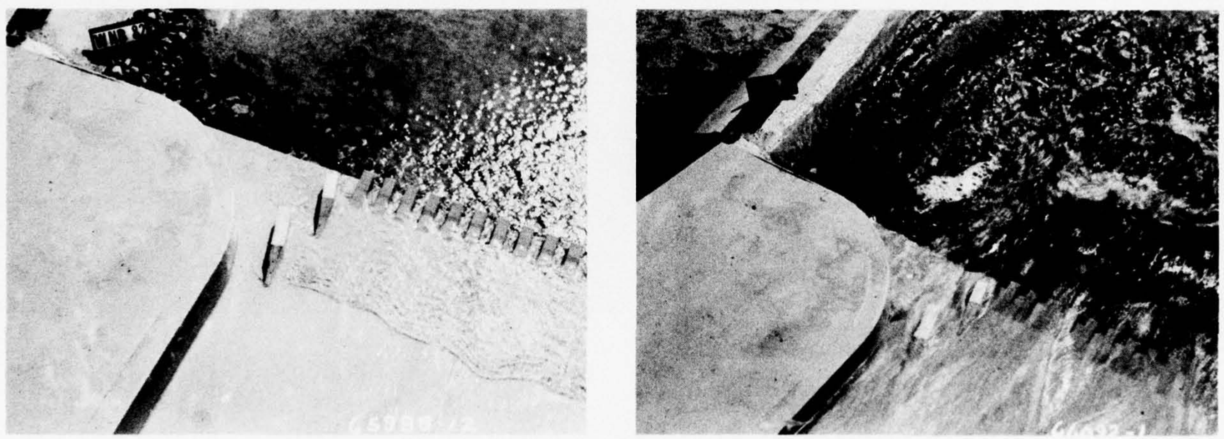


Looking upstream from right side

Photo 29. Spillway design 2; scour below spillway after hydrograph run



Four large dentates, two 10 ft high and two 12 ft high



Two 8-ft vertical blocks, 30 and 36 ft long

Figure 30. Replacement of six original 5- by 8-ft-high dentates. Left photos, gates closed; right photos, discharge 250,000 cfs. Note counterclockwise eddy

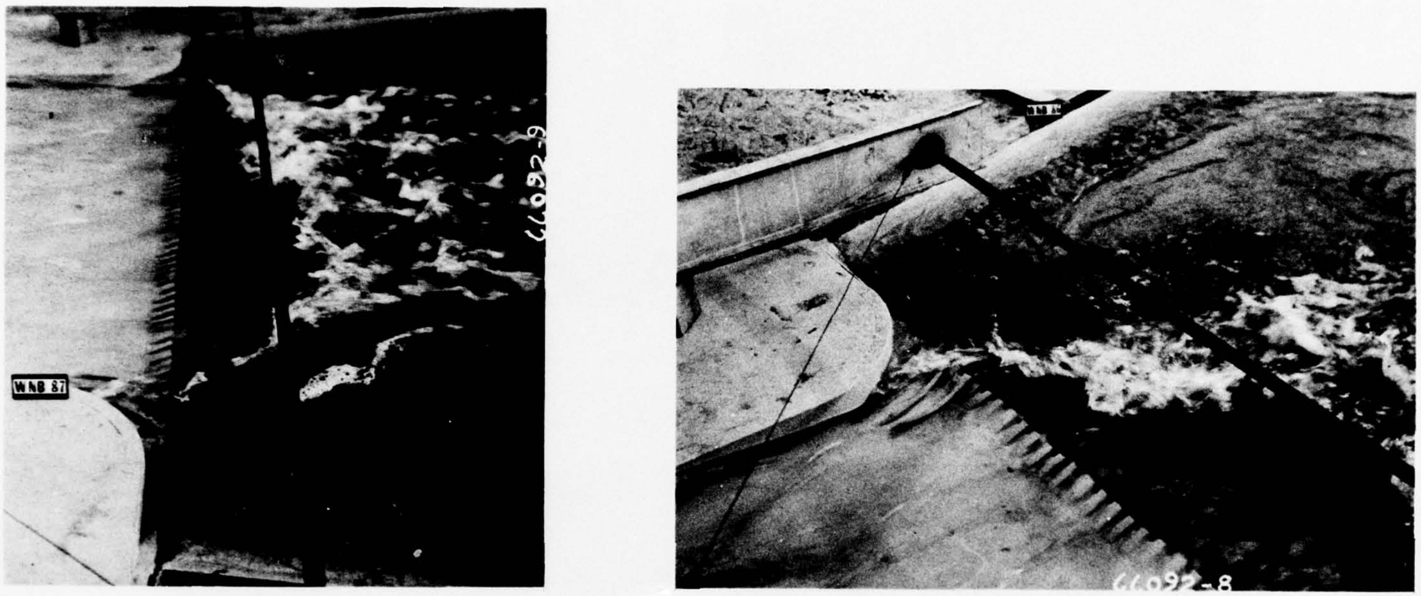
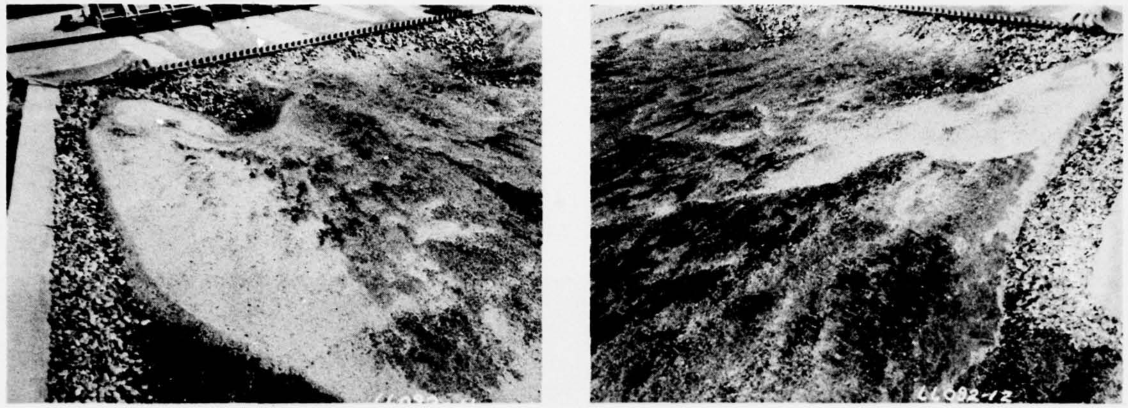
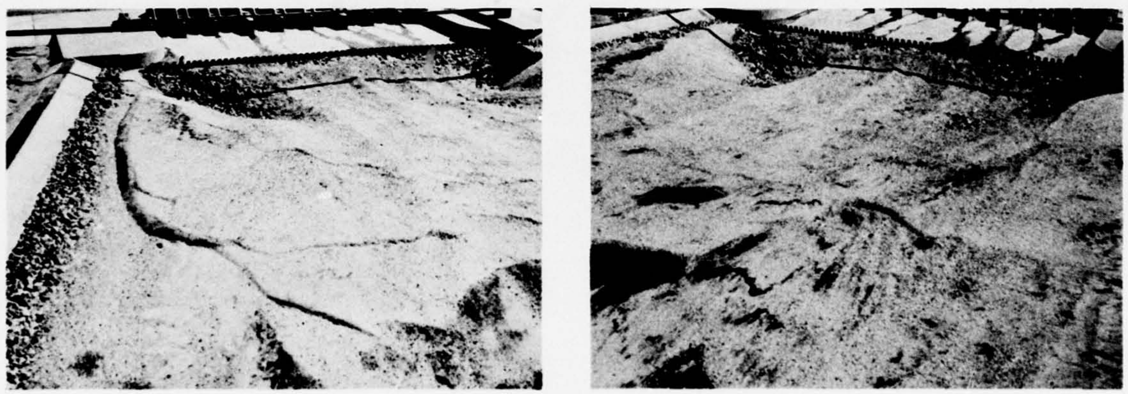


Photo 31. Hydraulic action with added curved guide vanes (design 1); discharge 250,000 cfs



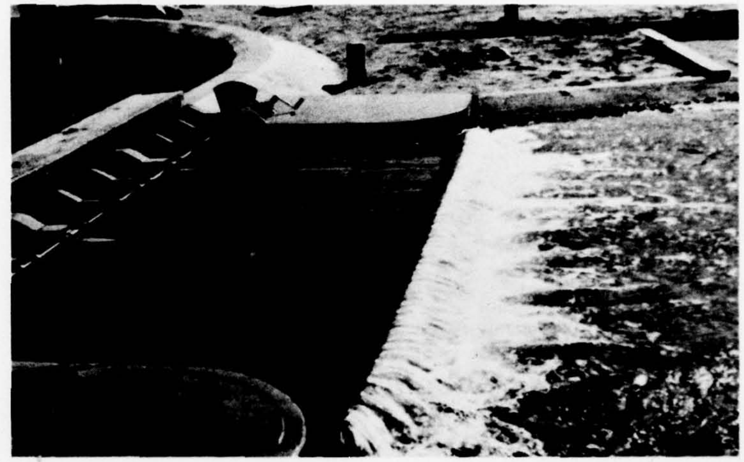
Design 1



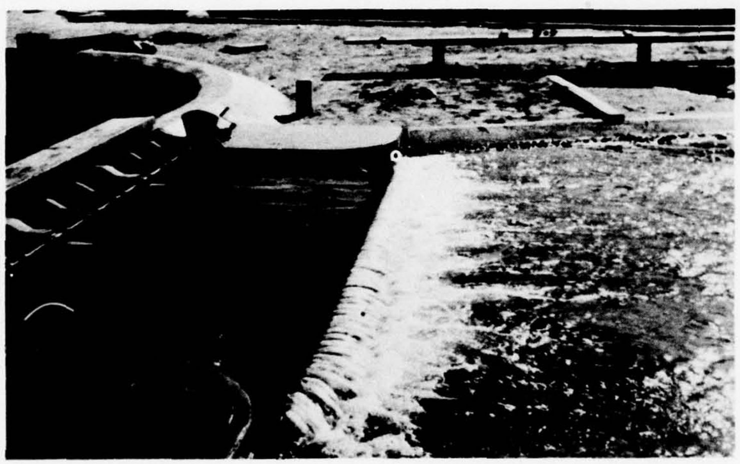
Design 2

Photo 32. Curved guide vanes, designs 1 and 2. Scour after 30-min run, discharge 250,000 cfs; left photos, looking upstream from right side; right photos, looking upstream from left side

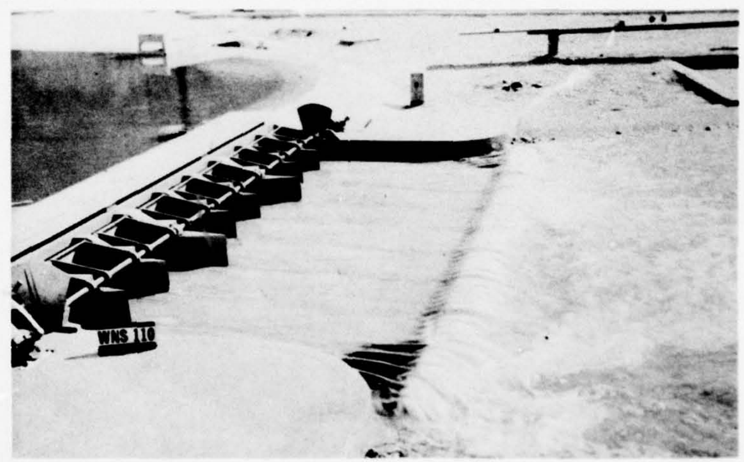
40,000 cfs



70,000 cfs

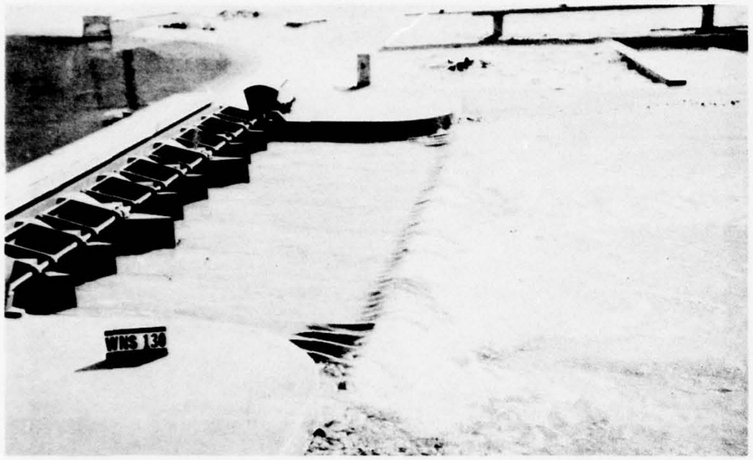


90,000 cfs

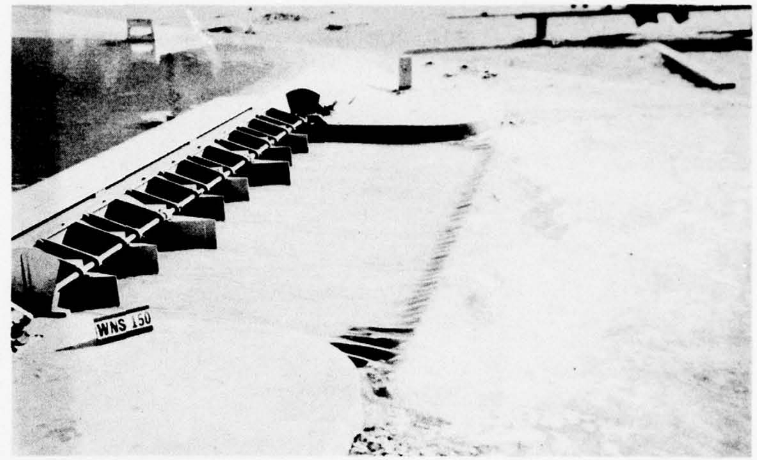


110,000 cfs

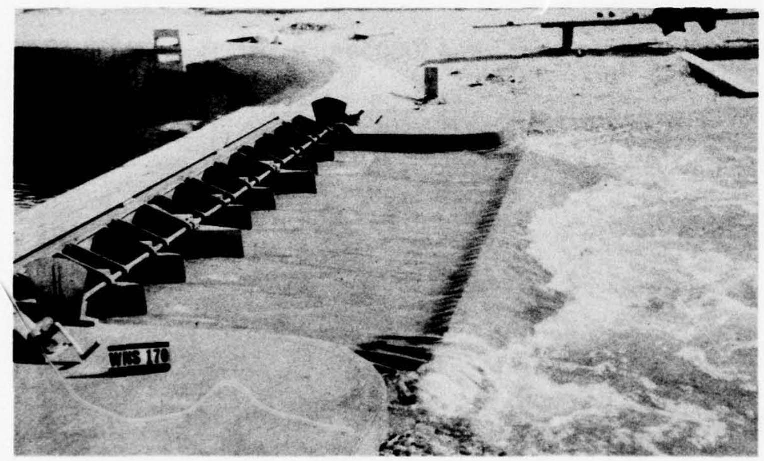
Photo 33. Final spillway design, design-flood hydrograph (sheet 1 of 3)



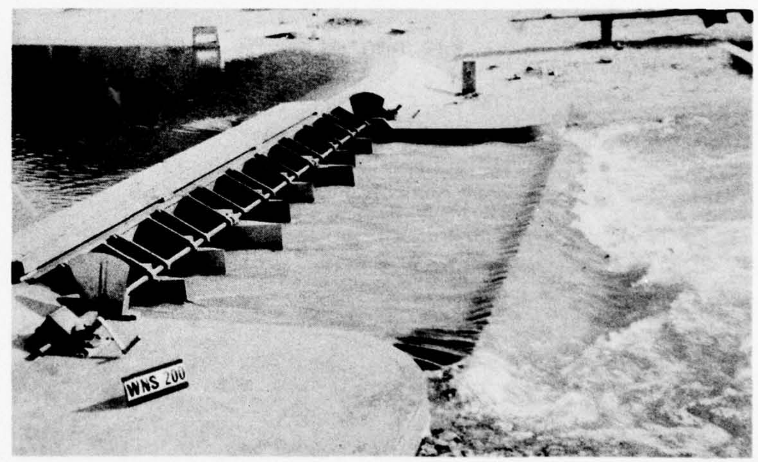
130,000 cfs



150,000 cfs

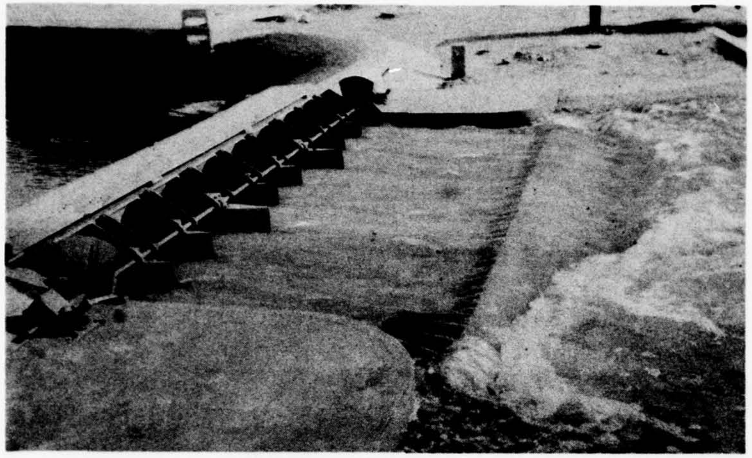


170,000 cfs

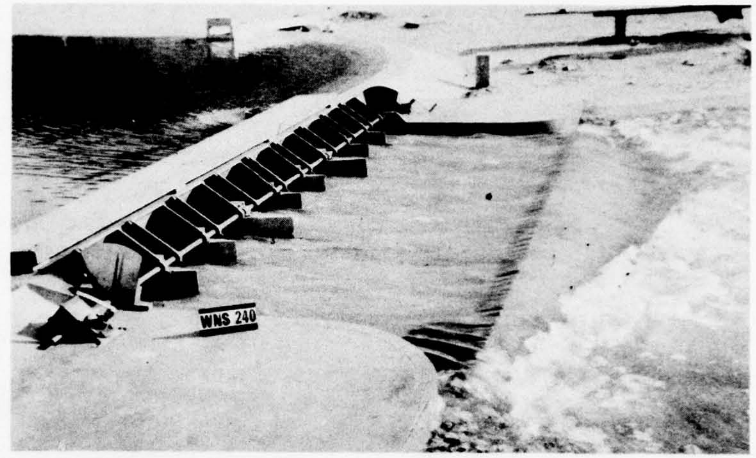


200,000 cfs

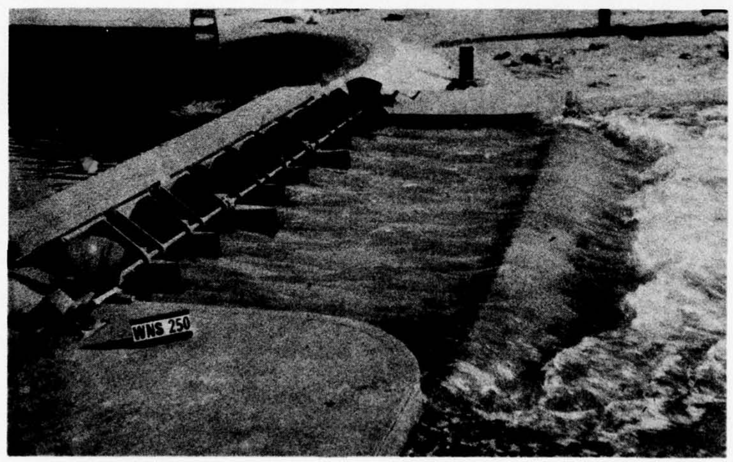
Photo 33 (sheet 2 of 3)



220,000 cfs

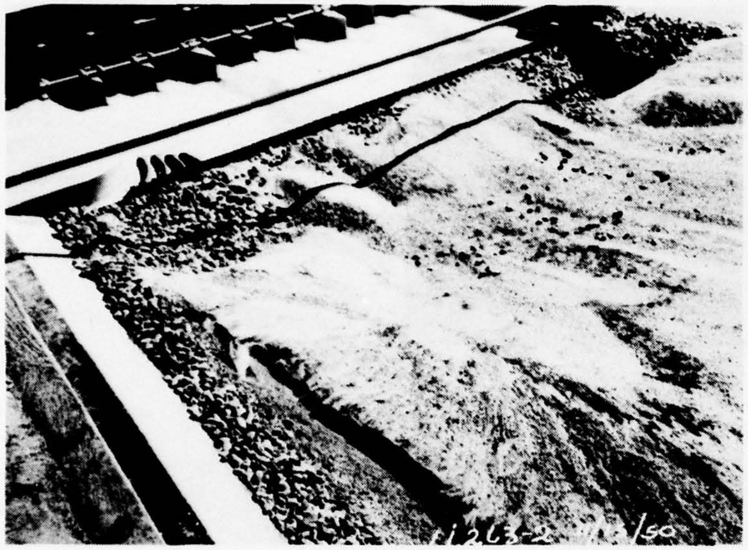


240,000 cfs



250,000 cfs

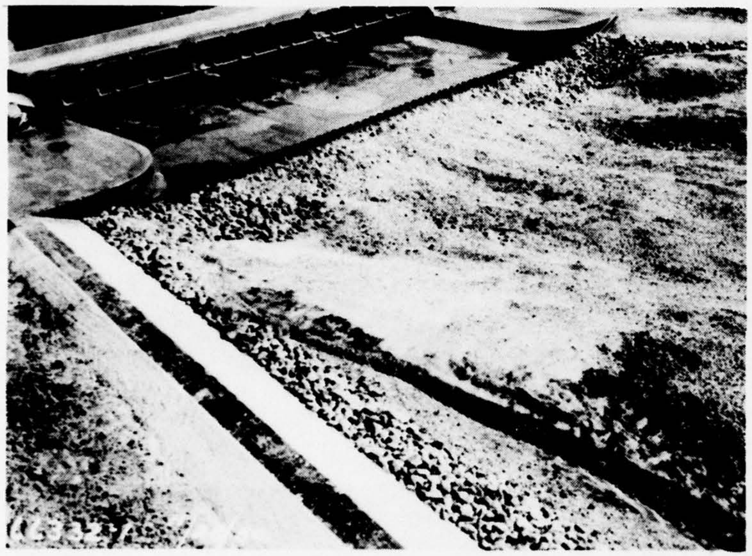
Photo 33 (sheet 3 of 3)



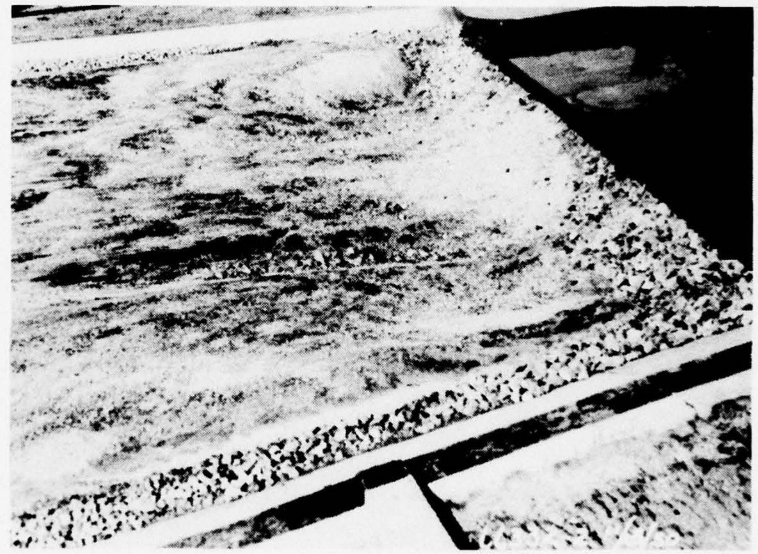
Looking upstream from right side

Looking upstream from left side

Photo 34. Curved guide vanes (final design). Type 1 rock protection, scour after hydrograph run



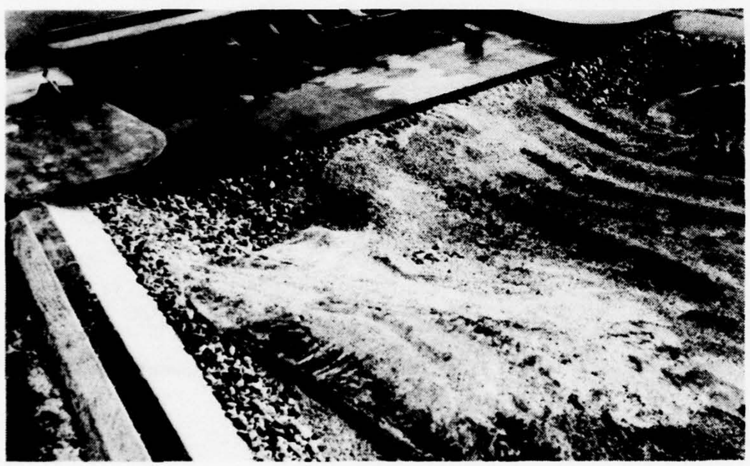
Looking upstream from right side

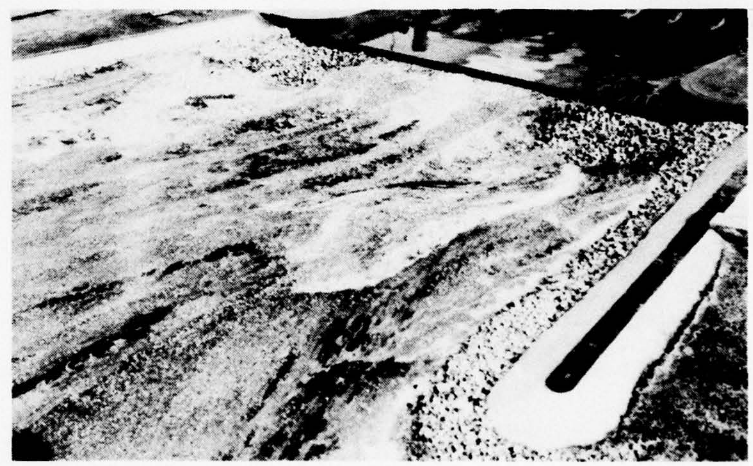


Looking across channel from left side

Photo 35. Curved guide vanes (final design). Type 2 rock protection, scour after partial hydrograph run

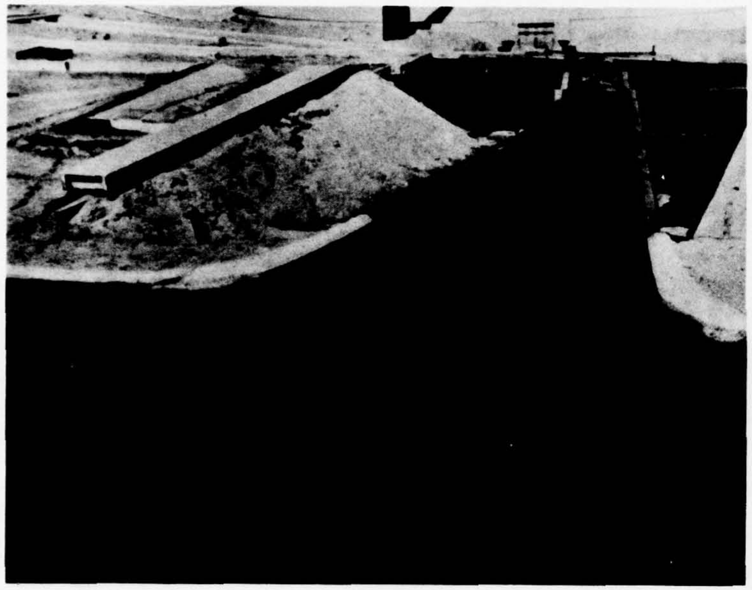
Looking upstream from right side





Looking upstream from left side

Photo 36. Curved guide vanes (final design). Type 2 rock protection, scour after completed hydrograph run

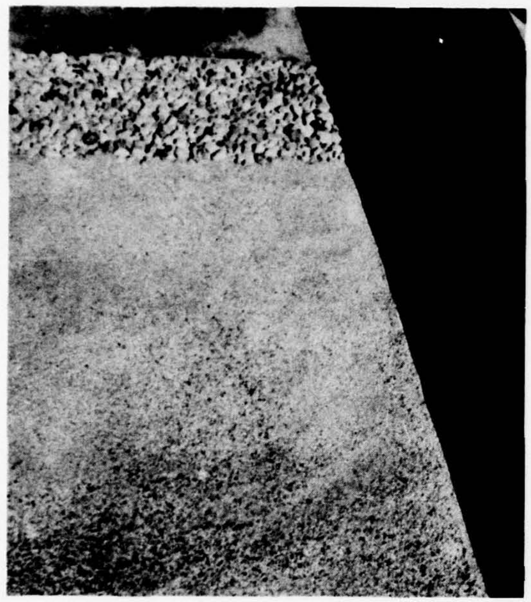


Looking downstream

Looking upstream

Photo 37. Single bay model, discharge 29,000 cfs

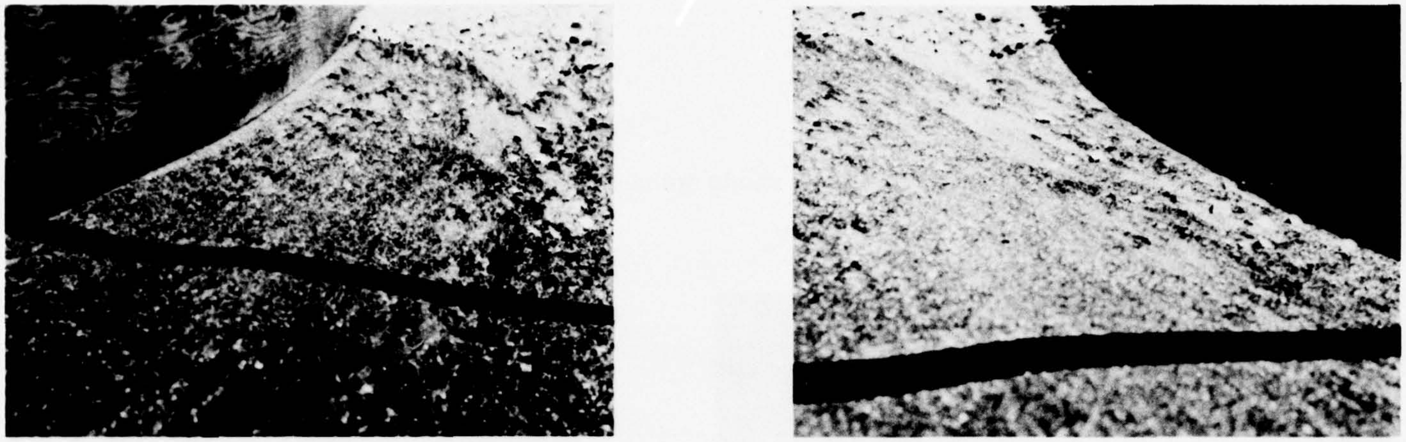
Photo 38. Single bay model; normal sill without rock apron. Scour after 1.5-hr run, looking downstream



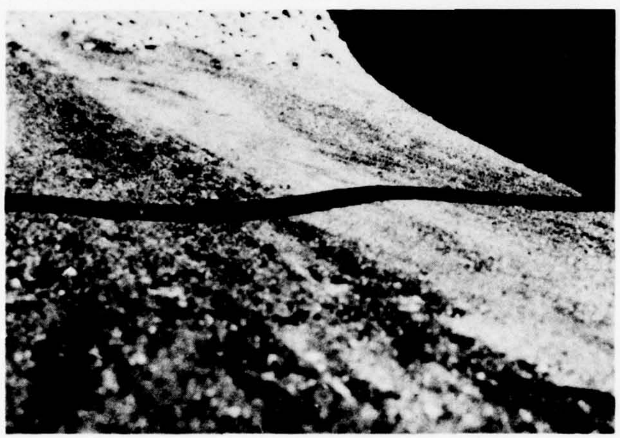
Normal sill

Skewed sill

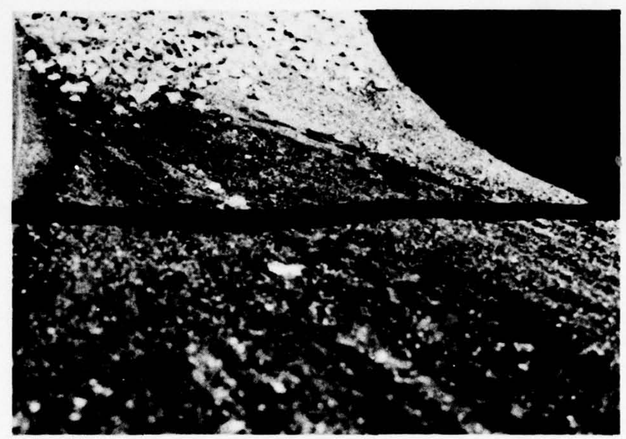
Photo 39. Single bay model, rock apron 25 ft wide by 3 ft thick



Normal sill, scour after 1.5-hr run

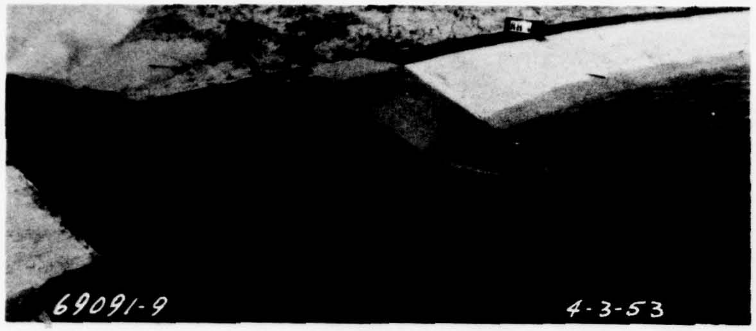


Skewed sill, scour after 1.5-hr run

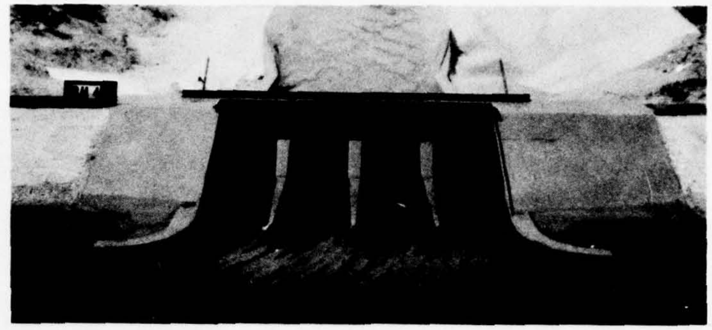


Skewed sill, scour after 4-hr run

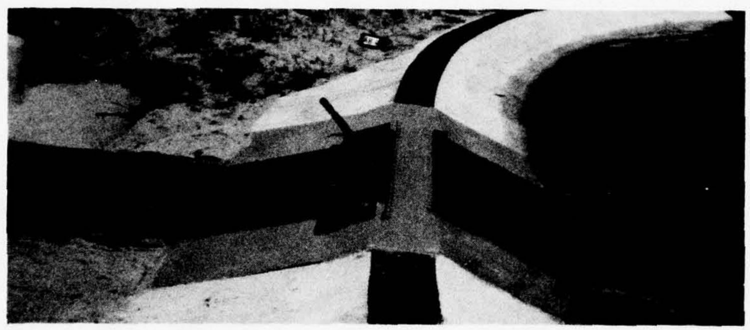
Photo 40. Single bay model, 25-ft-wide by 3-ft-thick rock apron, looking downstream



View from left side

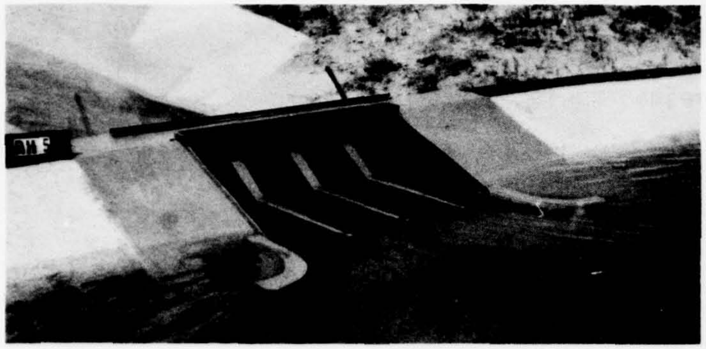


Looking downstream

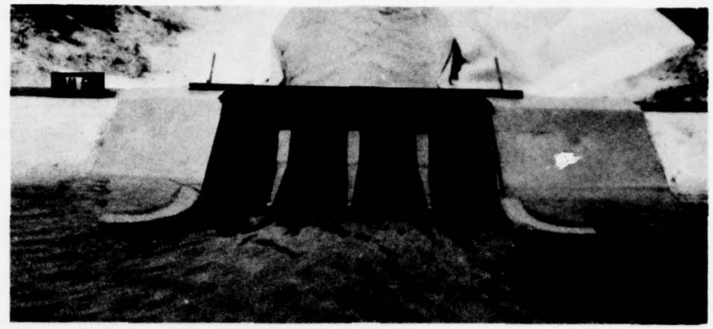


Looking along center line of dam from left to right

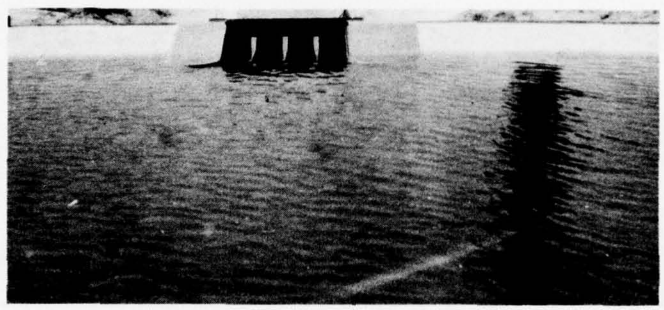
Photo 41. General outlet model, scale 1:60, discharge 40,000 cfs



View from left side

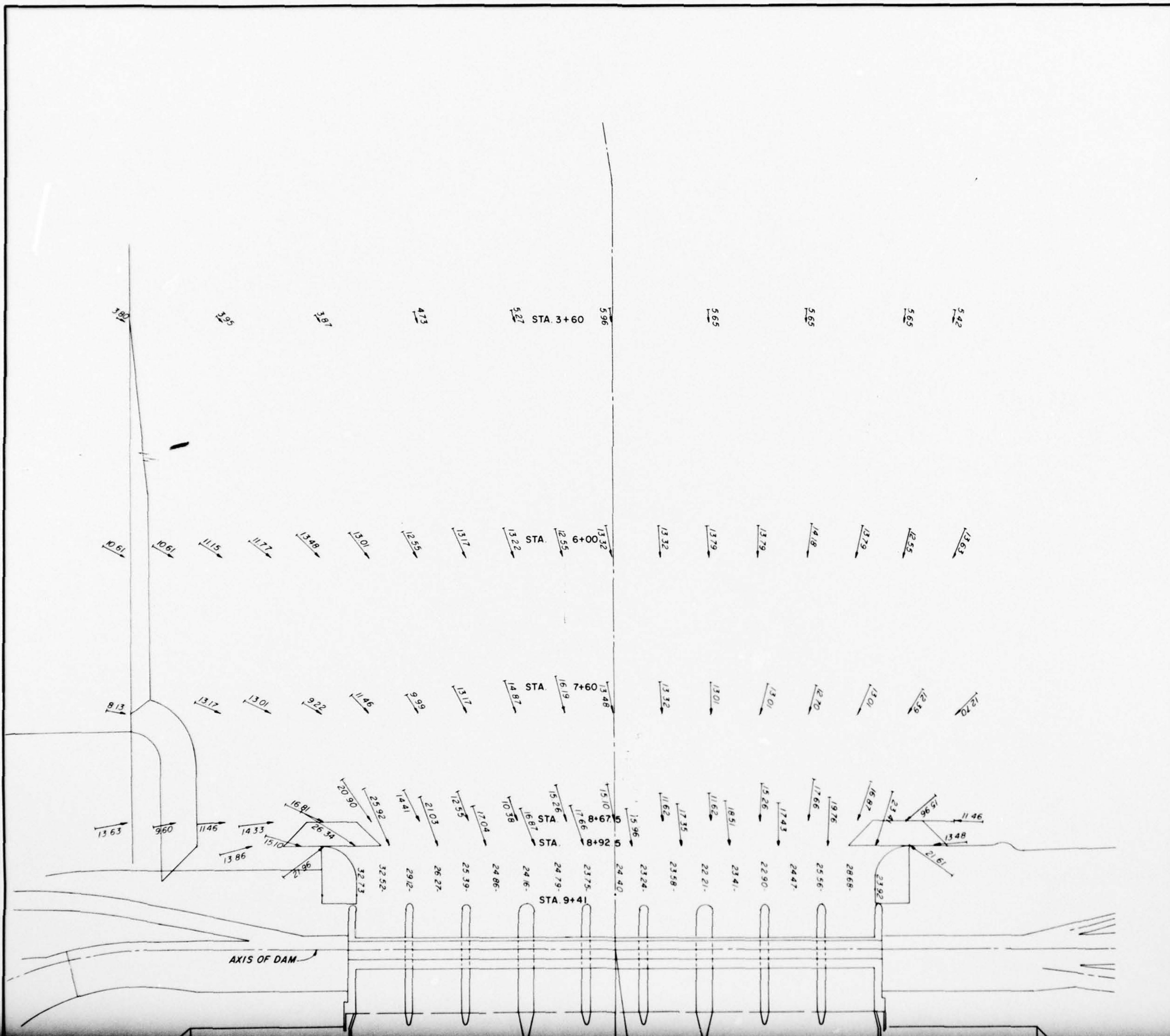


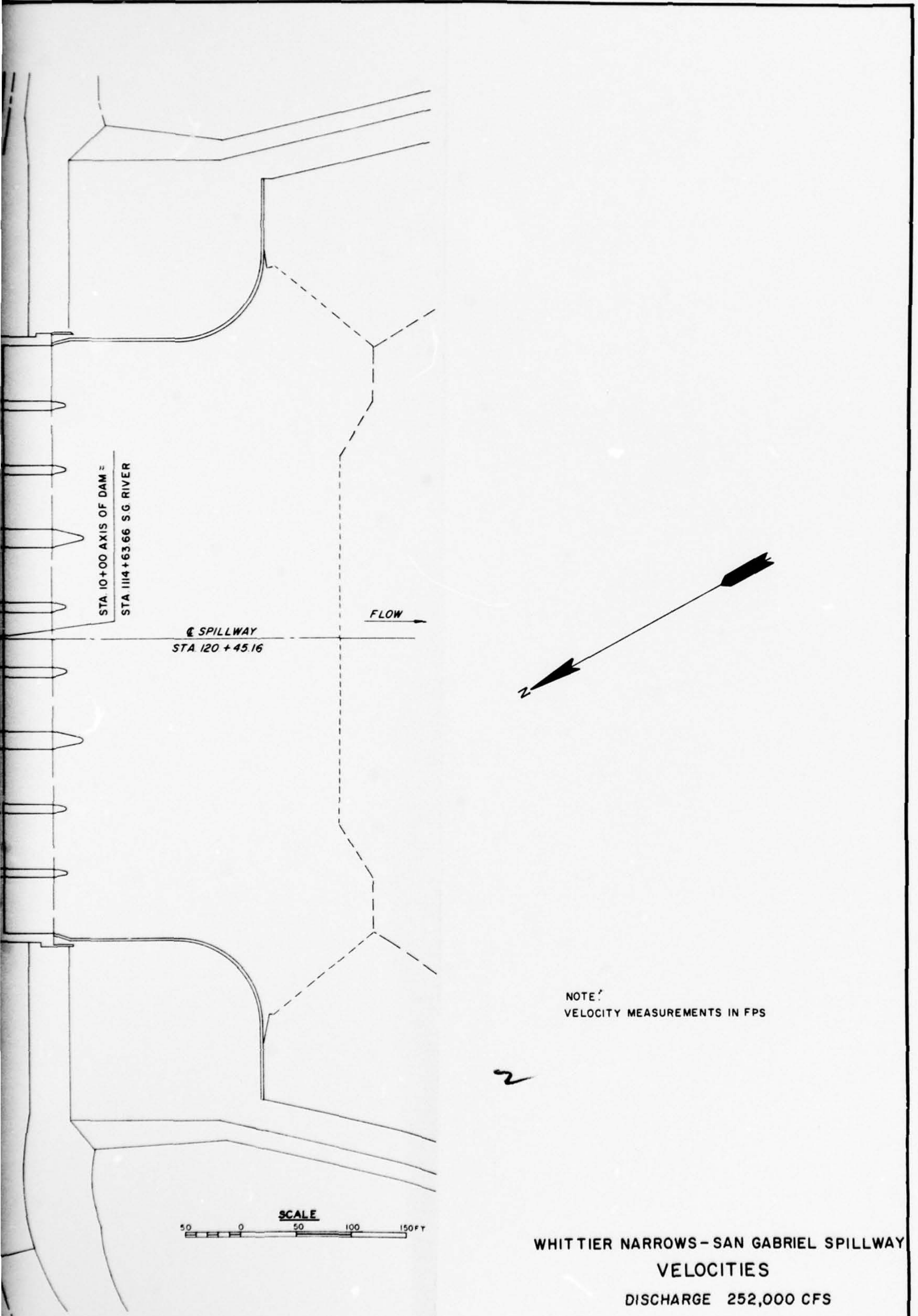
Looking downstream



Distant view looking downstream

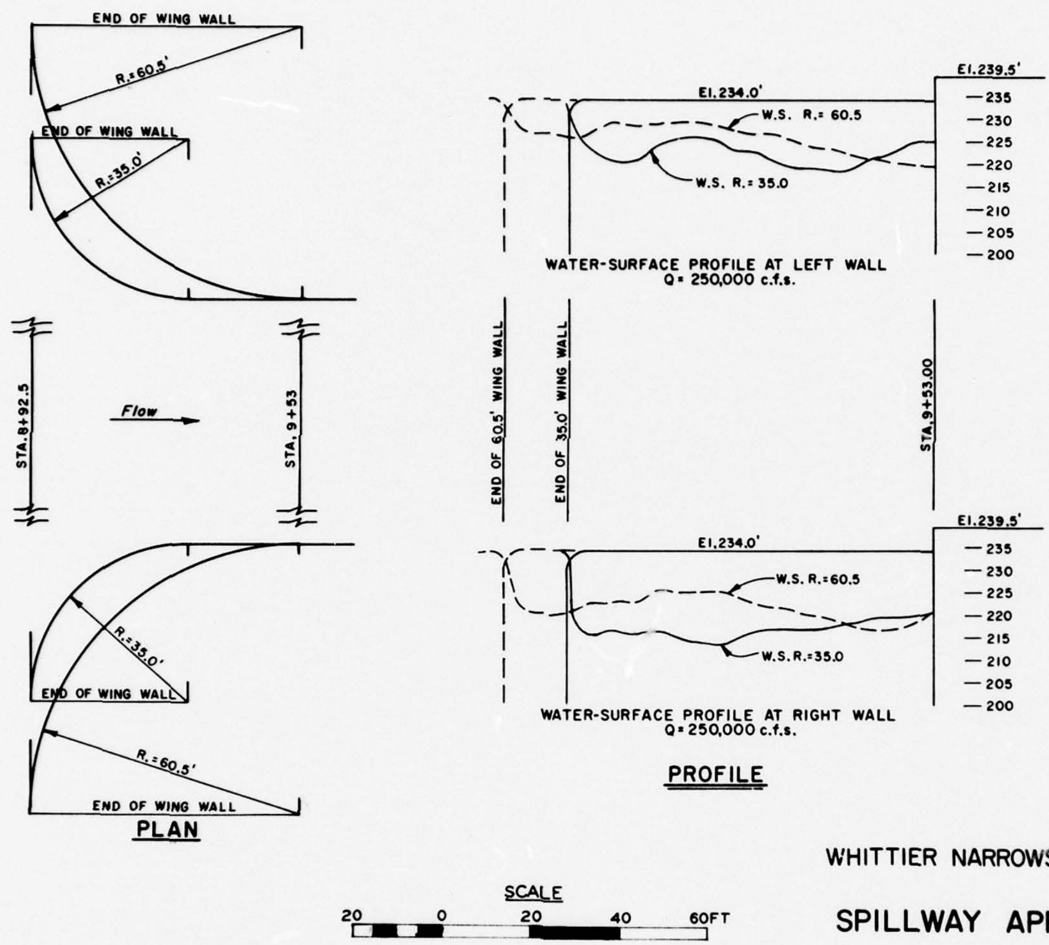
Photo 42. General outlet model, scale 1:60, discharge 50,000 cfs


STA. 3+60
STA. 6+00
STA. 7+60
STA. 8+67.5
STA. 8+92.5
STA. 9+41
AXIS OF DAM



NOTE:
VELOCITY MEASUREMENTS IN FPS

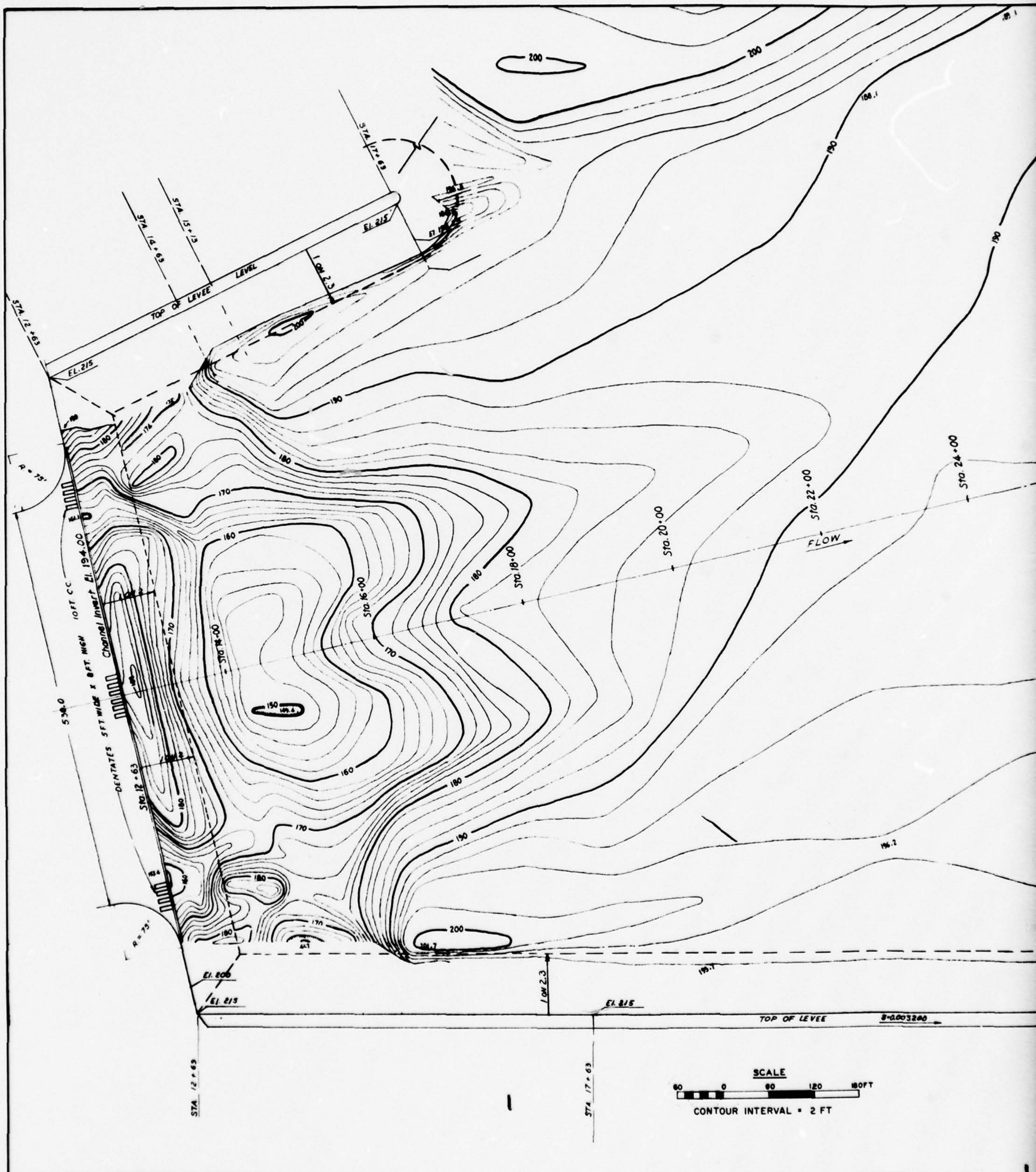
WHITTIER NARROWS - SAN GABRIEL SPILLWAY
VELOCITIES
DISCHARGE 252,000 CFS



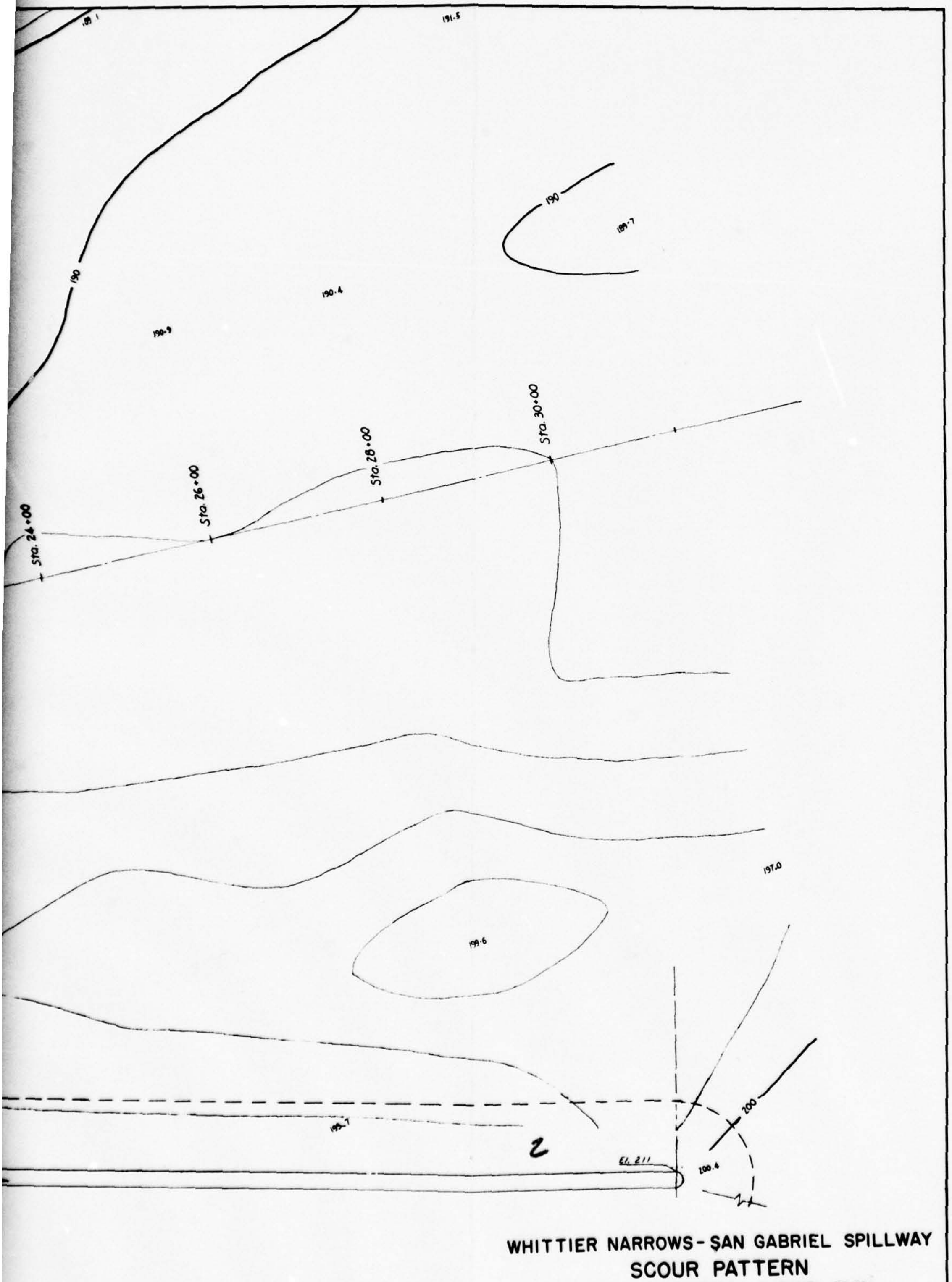
WHITTIER NARROWS-SAN GABRIEL SPILLWAY

SPILLWAY APPROACH CHANNEL

WATER SURFACE ALONG WING WALLS

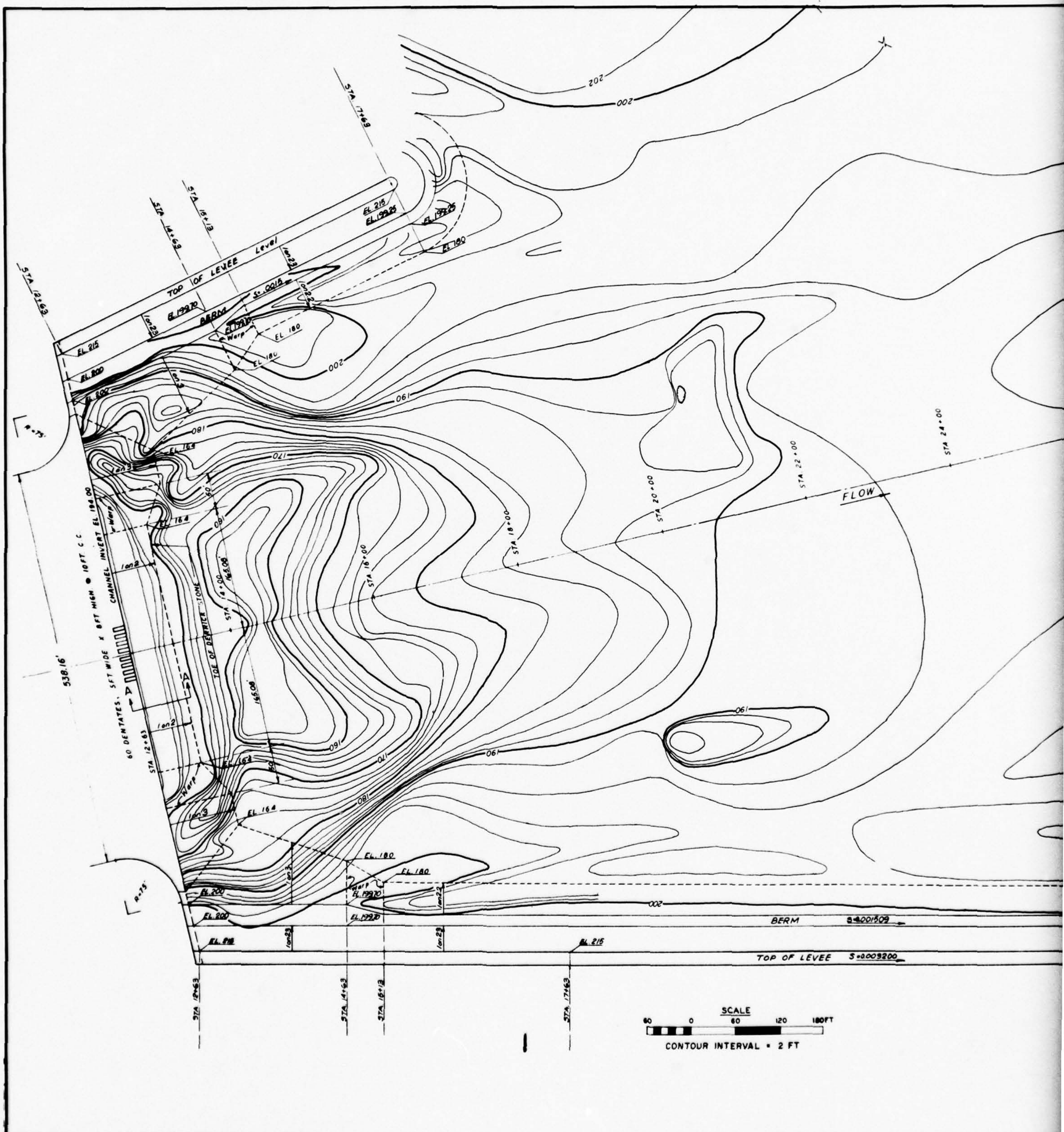


TOP OF LEVEE LEVEL
EL 215
STA 12+63
STA 14+63
STA 15+13
STA 17+63
1 ON 2.3
Channel Invert El 194.00
DENTATES 5 FT WIDE X 8 FT HIGH 10 FT C C
530.0
R = 75'
Sta. 12+63
Sta. 14+00
Sta. 16+00
Sta. 18+00
Sta. 20+00
Sta. 22+00
Sta. 24+00
FLOW
EL 200
EL 215
TOP OF LEVEE
SCALE
60 0 60 120 180 FT
CONTOUR INTERVAL = 2 FT

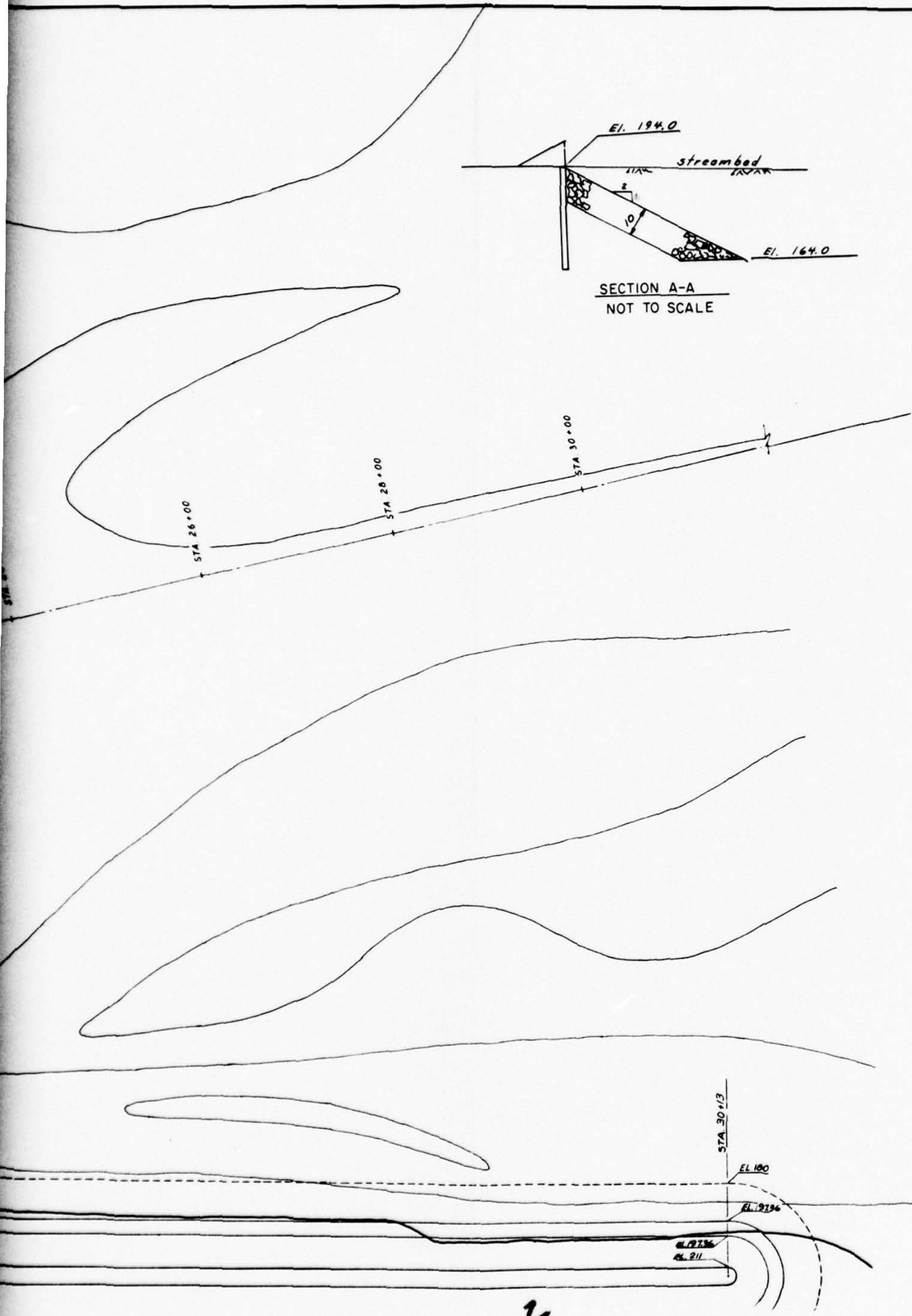


WHITTIER NARROWS - SAN GABRIEL SPILLWAY
SCOUR PATTERN
SCOUR IN DOWNSTREAM CHANNEL FROM
HYDROGRAPH RUN - PEAK 252,000 CFS
SPILLWAY DESIGN 1 (BASIC DESIGN)

PLATE 29



TOP OF LEVEE
BERM
FLOW
STA 12+63
STA 14+63
STA 16+12
STA 17+63
STA 18+00
STA 20+00
STA 22+00
STA 24+00
EL 215
EL 200
EL 180
EL 164
538.16'
60 DENTATES, 5FT WIDE X 8FT HIGH @ 10FT C.C.
CHANNEL INVERT EL 194.00
TOE OF BARRIER STONE
TOP OF LEVEE S=0.009200
SCALE
60 0 60 120 180FT
CONTOUR INTERVAL = 2 FT



WHITTIER NARROWS - SAN GABRIEL SPILLWAY

SCOUR PATTERN

SCOUR IN DOWNSTREAM CHANNEL FROM
HYDROGRAPH RUN - PEAK 252,000 CFS
SPILLWAY DESIGN 2 WITH ROCK PROTECTION TYPE 1

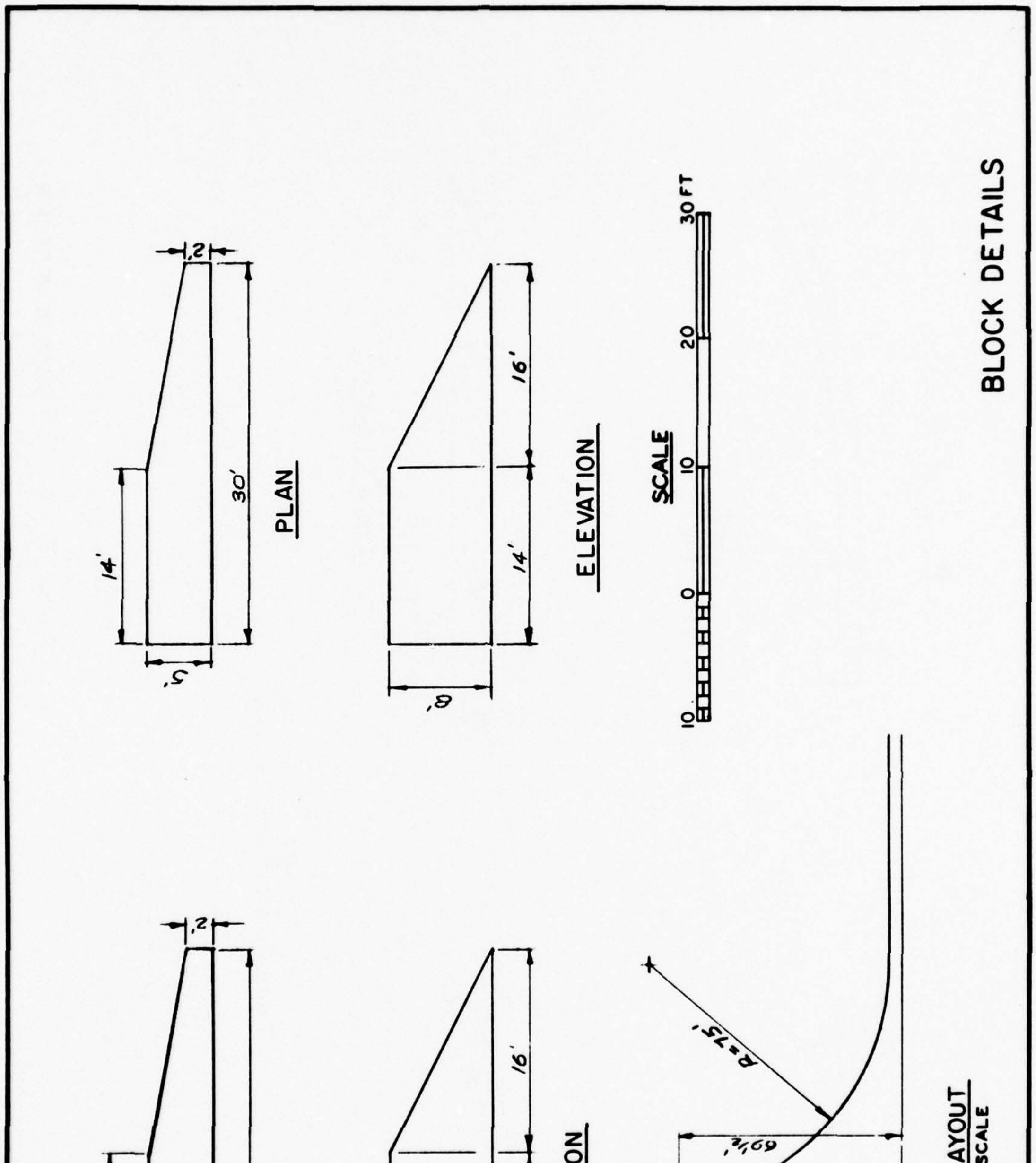
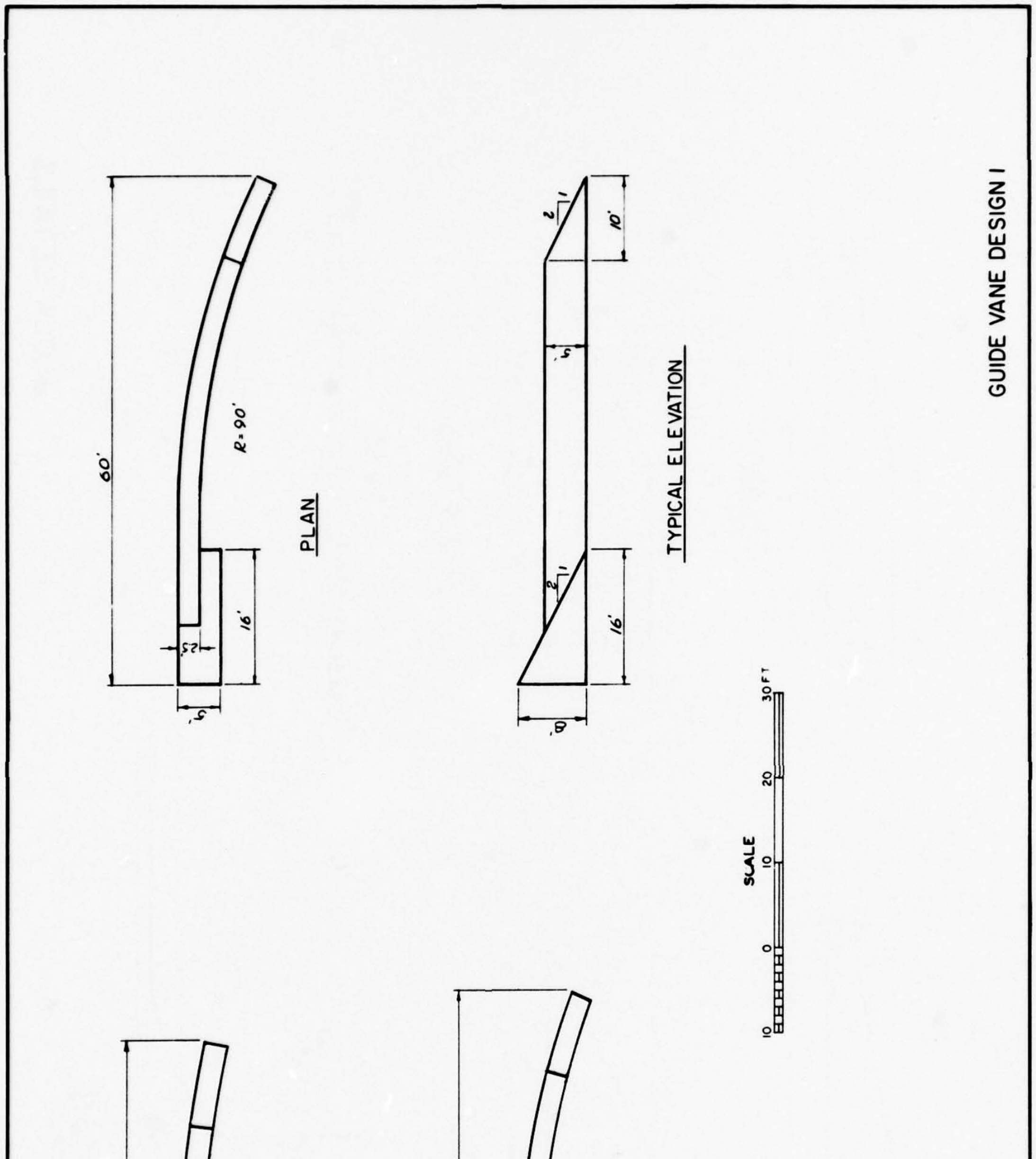
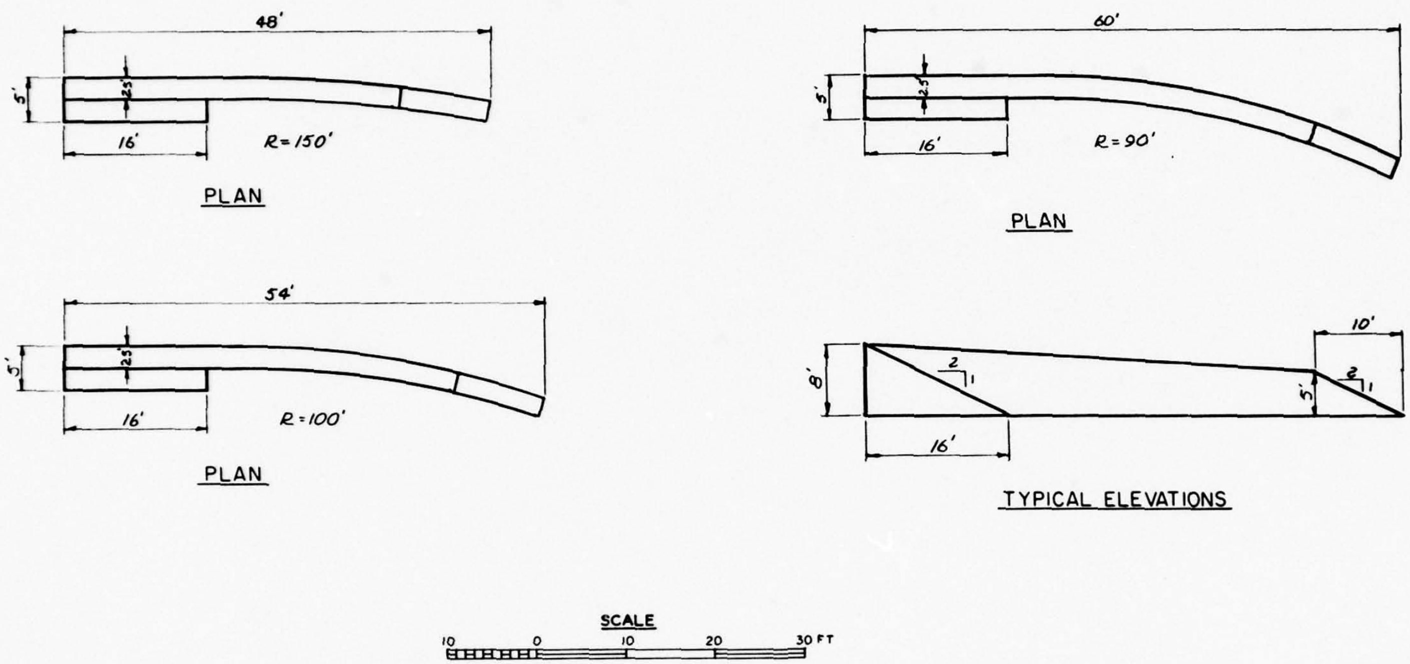

BLOCK DETAILS
PLAN
ELEVATION
PLAN
ELEVATION
SCALE
10
0
10
20
30 FT
BLOCK LAYOUT
NOT TO SCALE
BLOCKS
DENTATES
R=75'
69 1/2'
20'
36'
5'
2'
8'
16'
14'
30'

PLATE 31

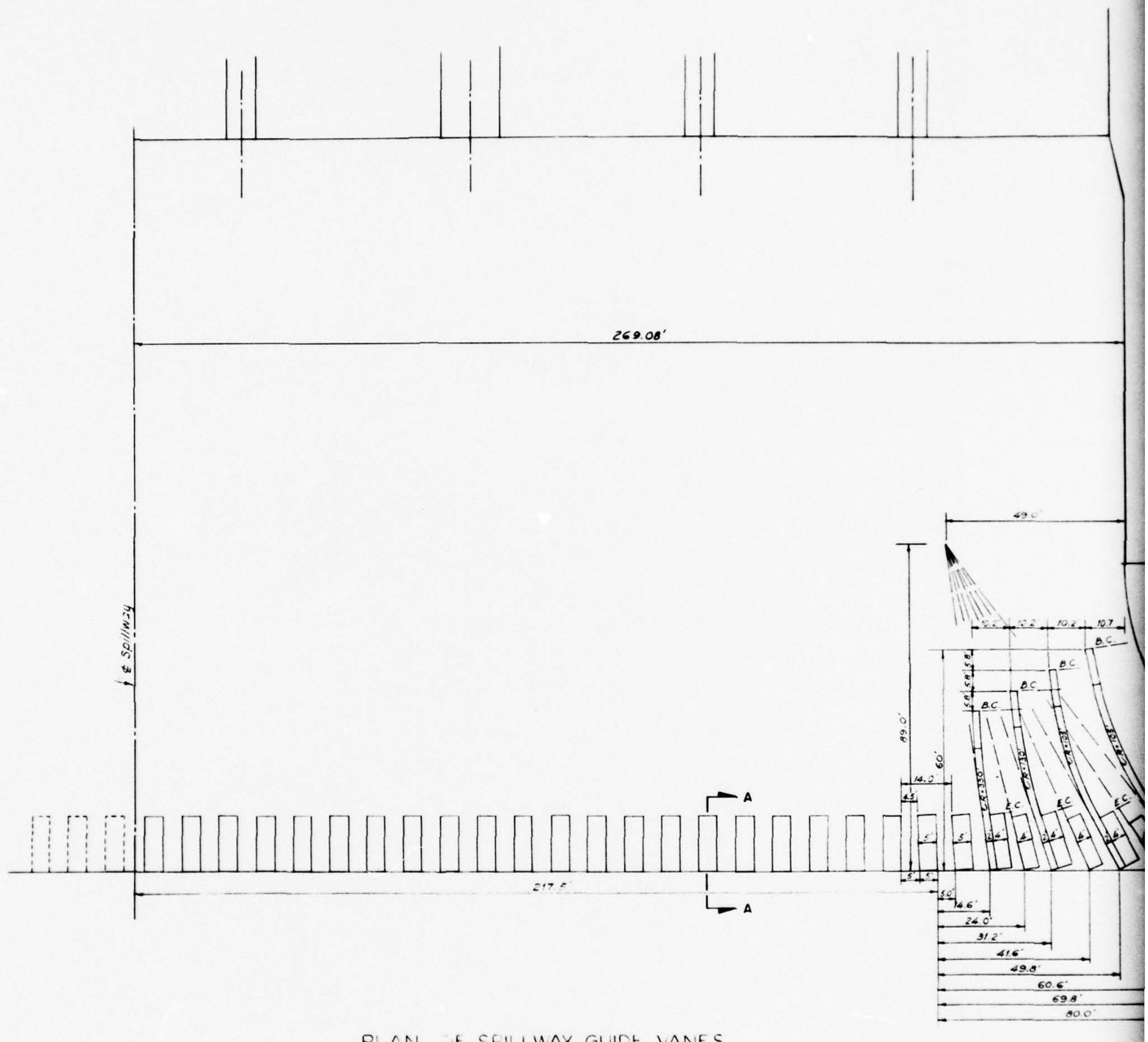


GUIDE VANE DESIGN I

PLATE 32



GUIDE VANE DESIGN 2

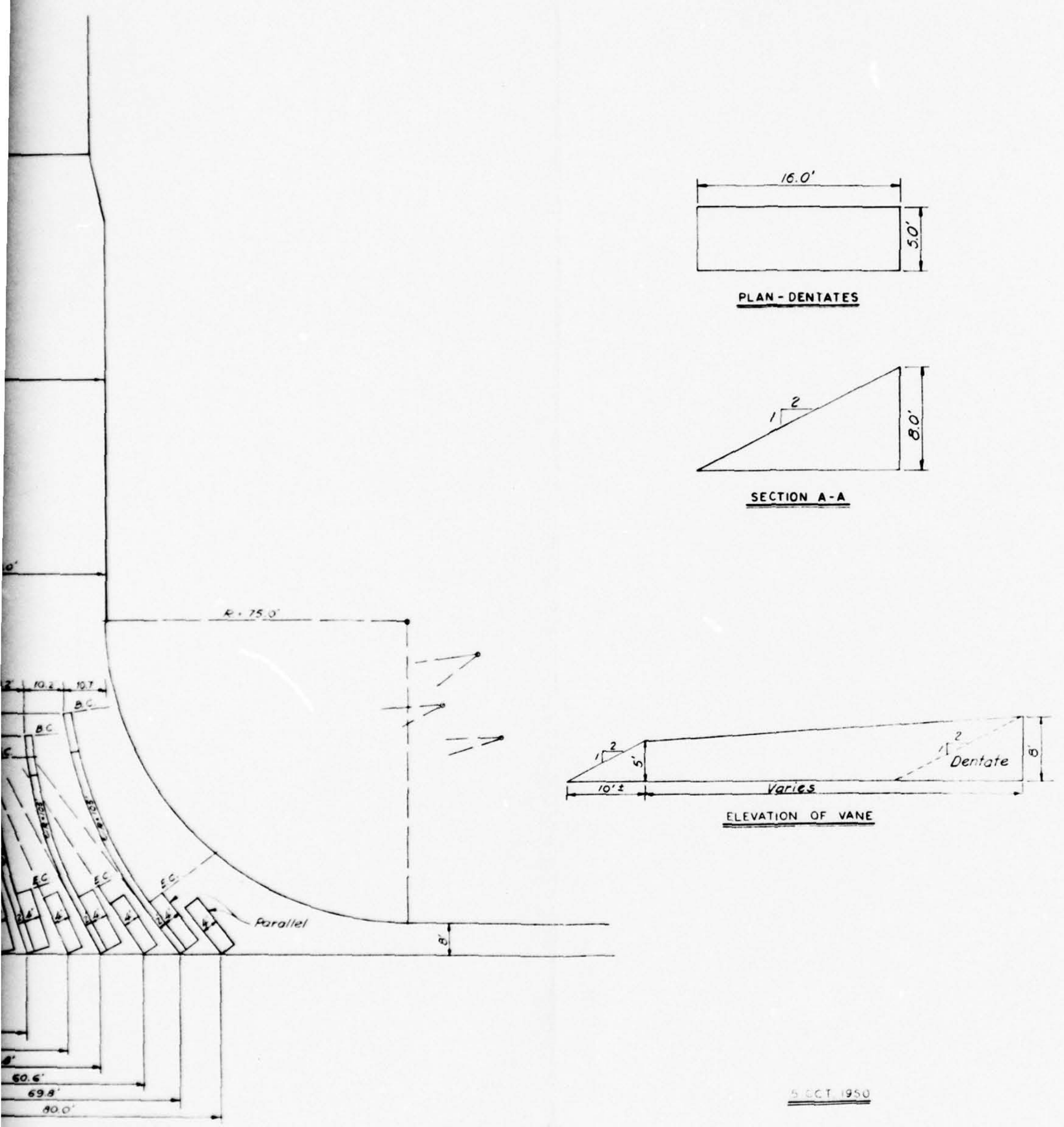


PLAN OF SPILLWAY GUIDE VANES

SCALE

10 0 10 20 30 40 FT

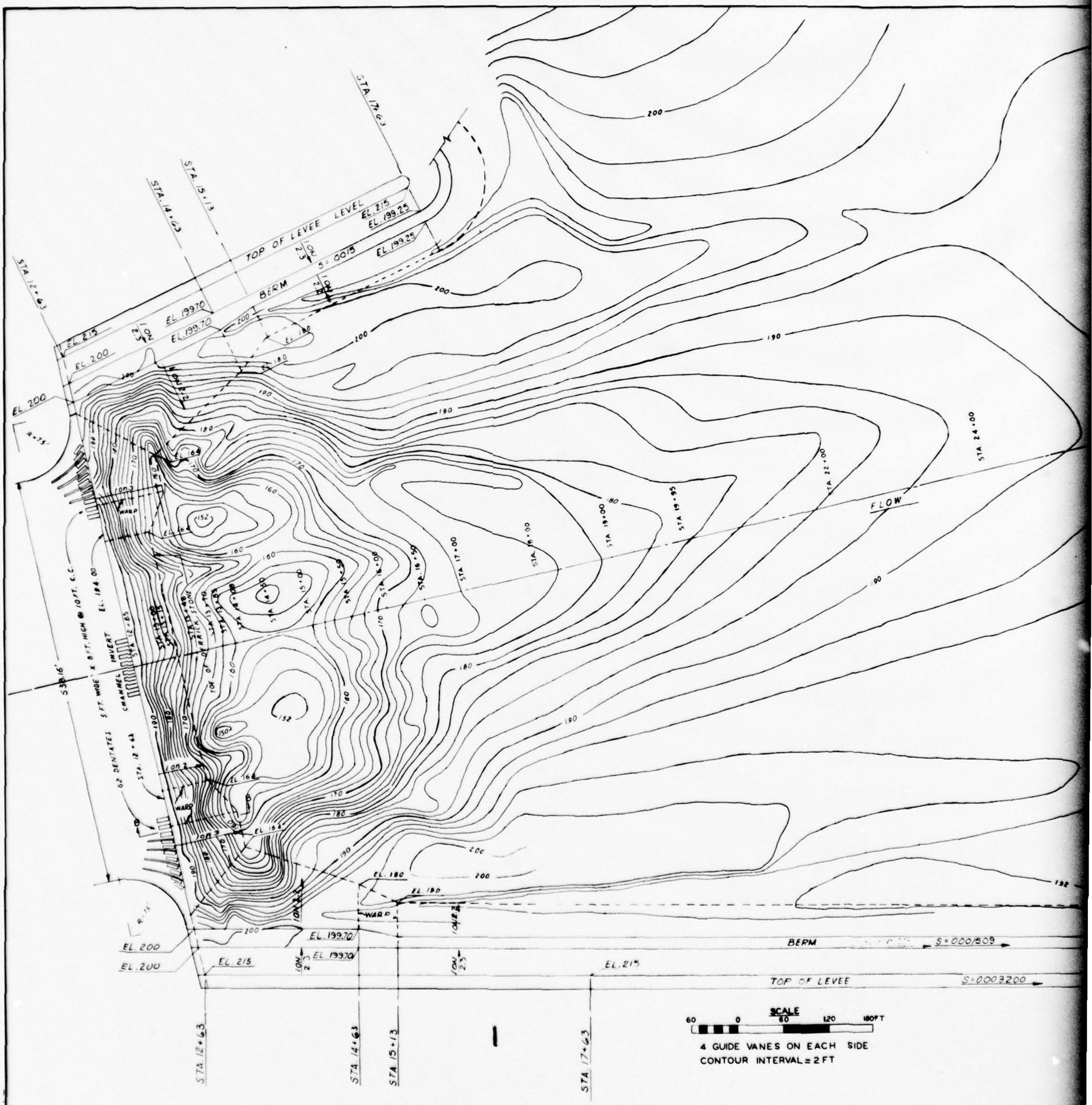
1

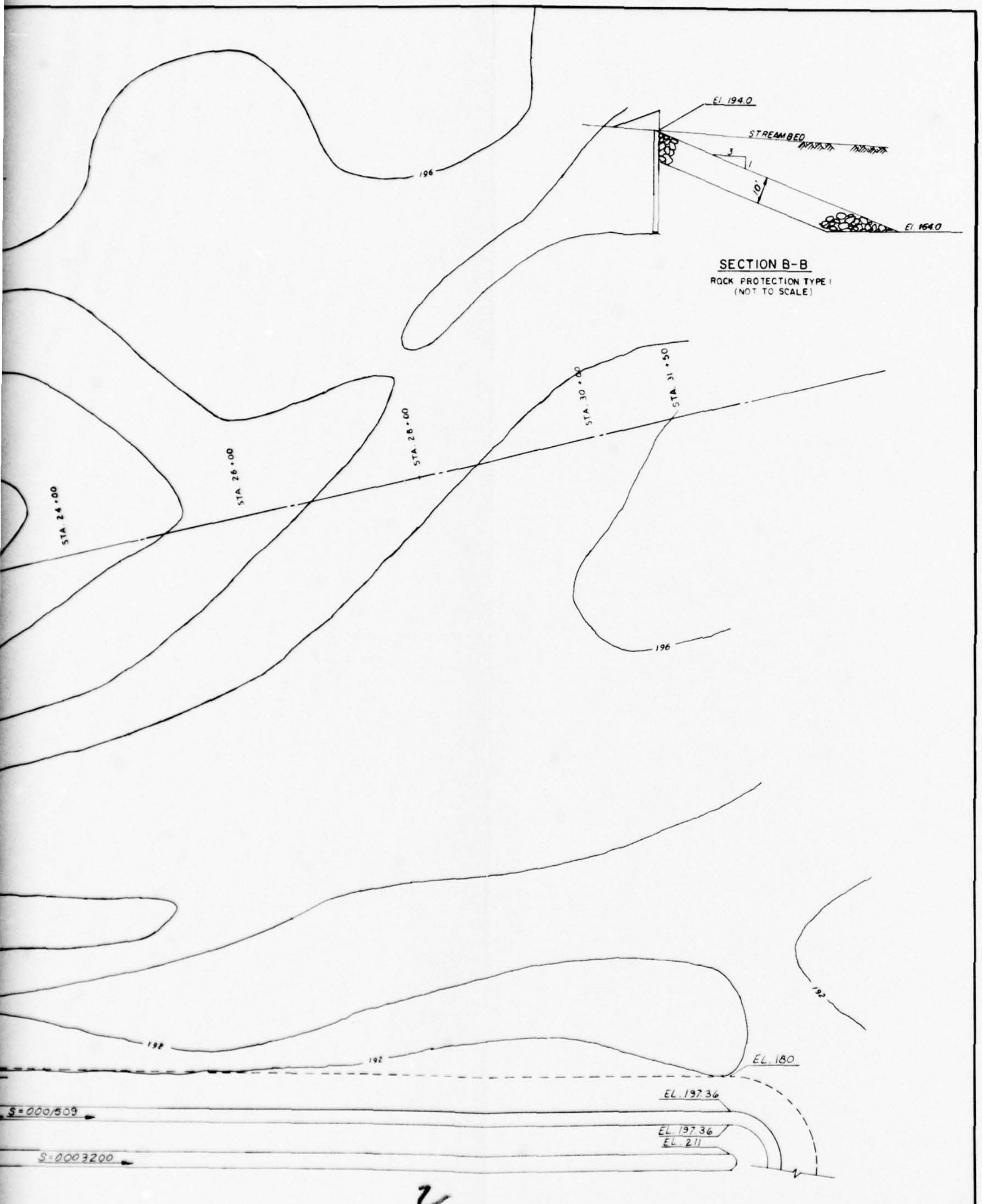


WHITTIER NARROWS - SAN GABRIEL SPILLWAY

DEFLECTOR VANE AND DENTATE DETAILS

FINAL DESIGN

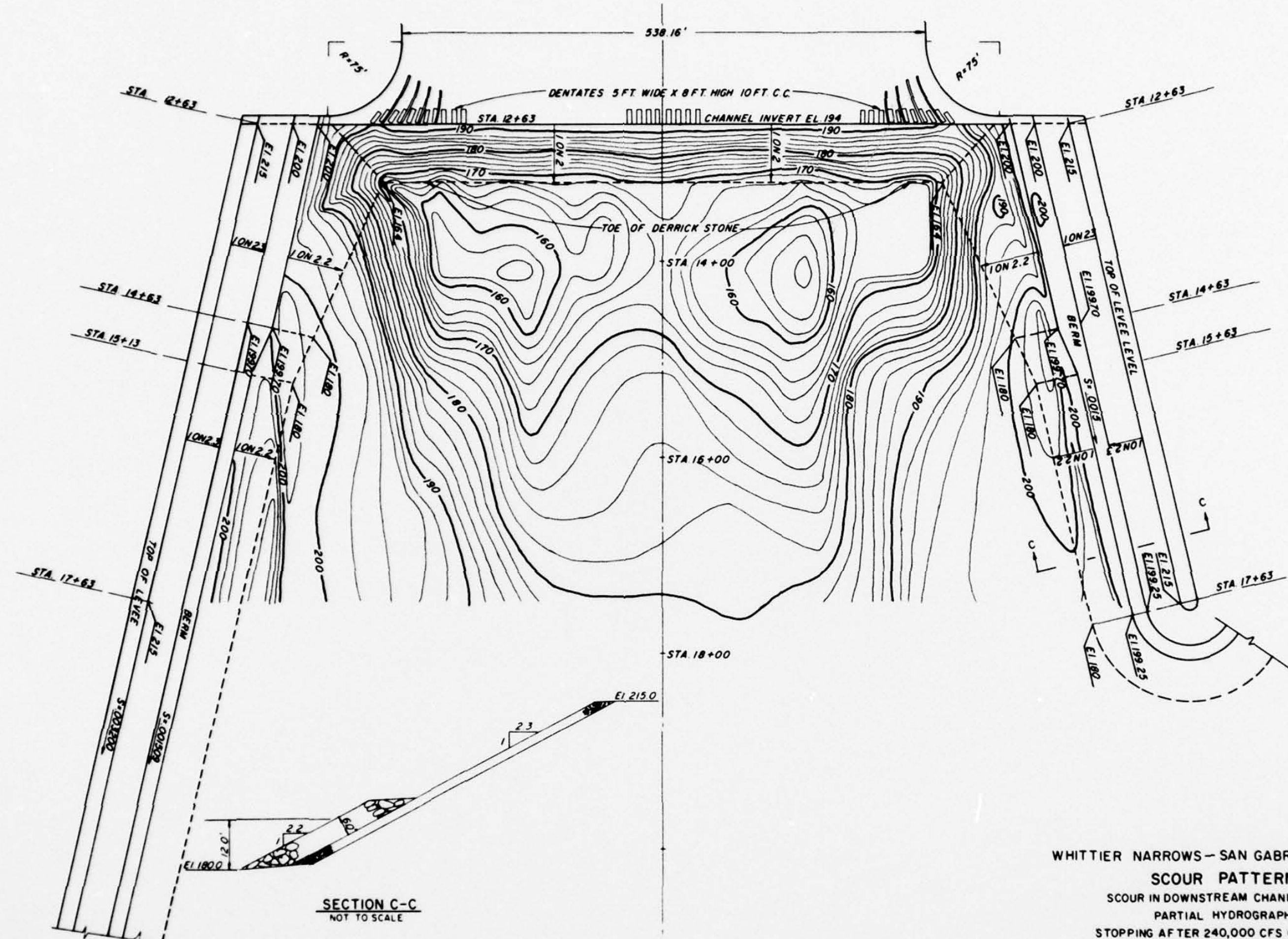

TOP OF LEVEE LEVEL
BERM
S=.0015
EL. 215
EL. 199.25
EL. 199.70
EL. 200
STA. 12+63
STA. 14+63
STA. 15+13
STA. 17+63
FLOW
62 DENTATES 5 FT. WIDE X 8 FT. HIGH @ 10 FT. C.C.
CHANNEL INVERT EL. 184.00
538.16'
WARD
STA. 17+00
STA. 18+00
STA. 19+00
STA. 19+95
STA. 22+00
STA. 24+00
TOP OF LEVEE
S=0.003200
S=0.001509
SCALE
60 0 60 120 180 FT
4 GUIDE VANES ON EACH SIDE
CONTOUR INTERVAL = 2 FT



WHITTIER NARROWS - SAN GABRIEL SPILLWAY

SCOUR PATTERN

SCOUR IN DOWNSTREAM CHANNEL FROM
HYDROGRAPH RUN - PEAK 252,000 CFS
FINAL SPILLWAY DESIGN WITH ROCK PROTECTION TYPE 1

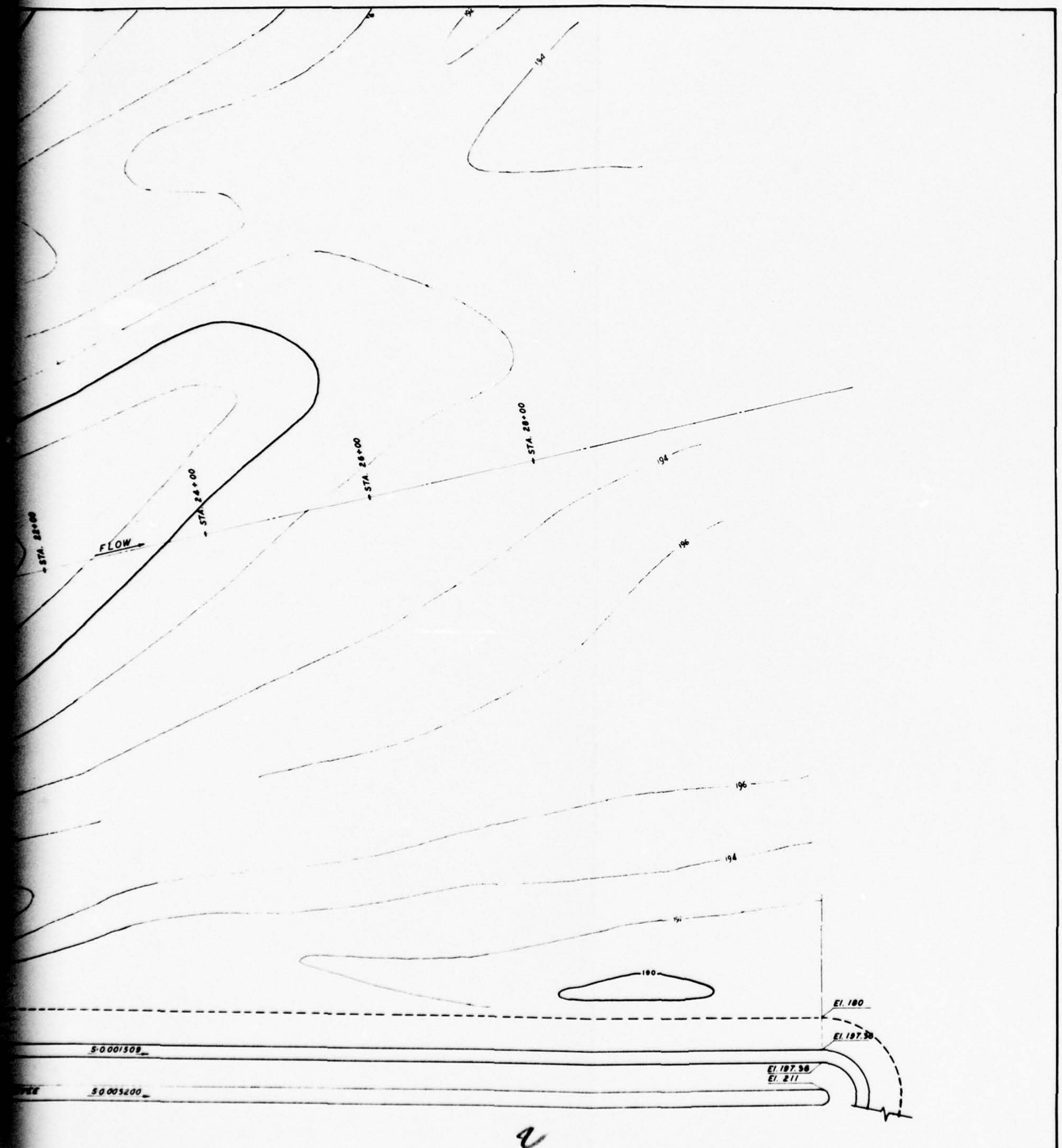


WHITTIER NARROWS – SAN GABRIEL SPILLWAY

SCOUR PATTERN

SCOUR IN DOWNSTREAM CHANNEL FROM
PARTIAL HYDROGRAPH
STOPPING AFTER 240,000 CFS ON RECEDING
SIDE OF HYDROGRAPH
FINAL SPILLWAY DESIGN WITH ROCK PROTECTION TYPE 2

TOP OF LEVEE
LEVEL
BERM
EL 215
EL 200
EL 180
FLOW
DENTATES 5 FT. WIDE X 8 FT. HIGH 10 FT. C.C.
TOE OF RIPRAP STONE
538.16'
R = 75'
STA. 12+63
STA. 14+63
STA. 15+13
STA. 17+63
STA. 18+00
STA. 20+00
STA. 22+00
STA. 24+00
S=0.001509
S=0.003200
SCALE
CONTOUR INTERVAL = 2 FT.

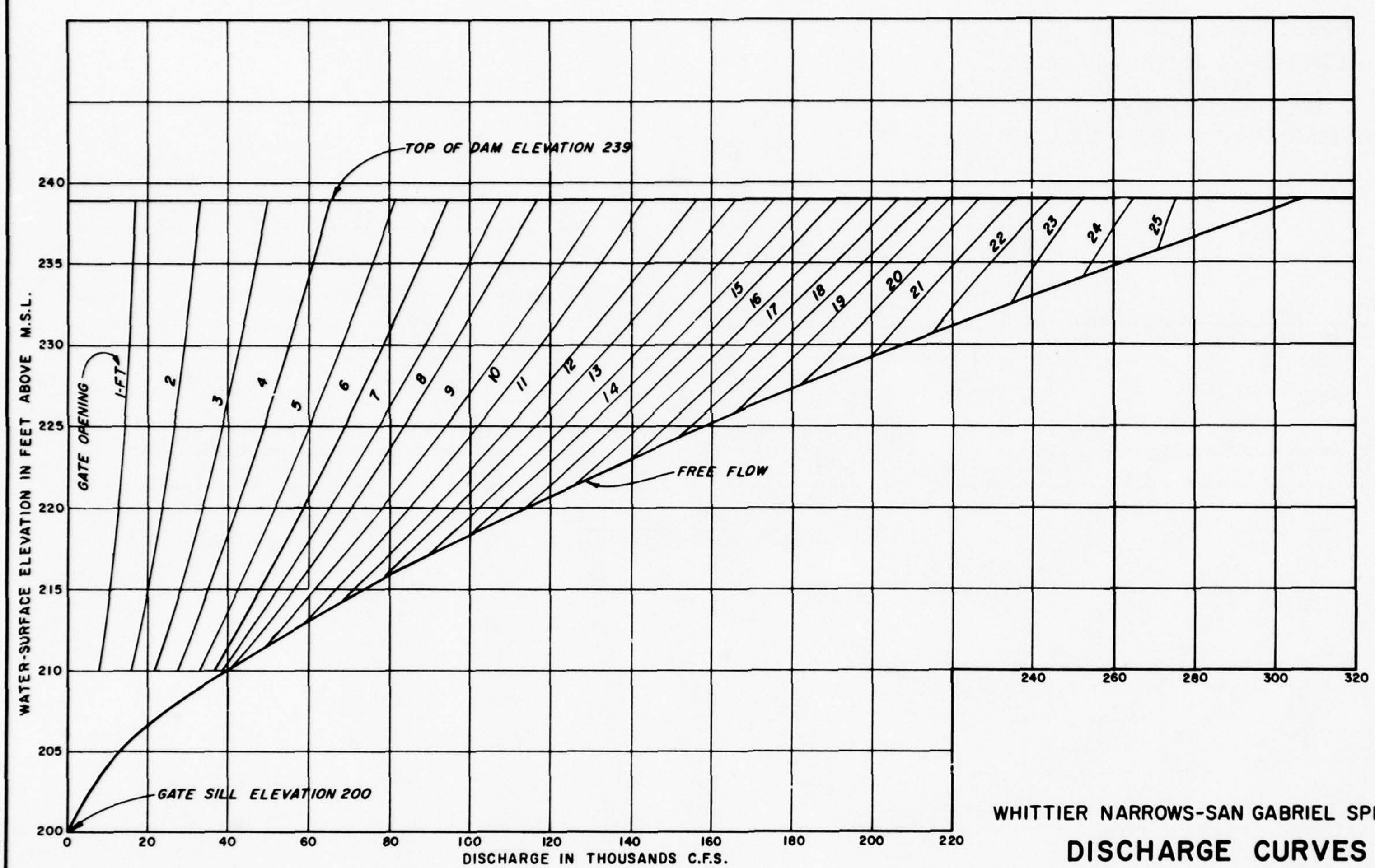


WHITTIER NARROWS - SAN GABRIEL SPILLWAY

SCOUR PATTERN

SCOUR IN DOWNSTREAM CHANNEL FROM
COMPLETED HYDROGRAPH
PEAK DISCHARGE OF 252,000 CFS

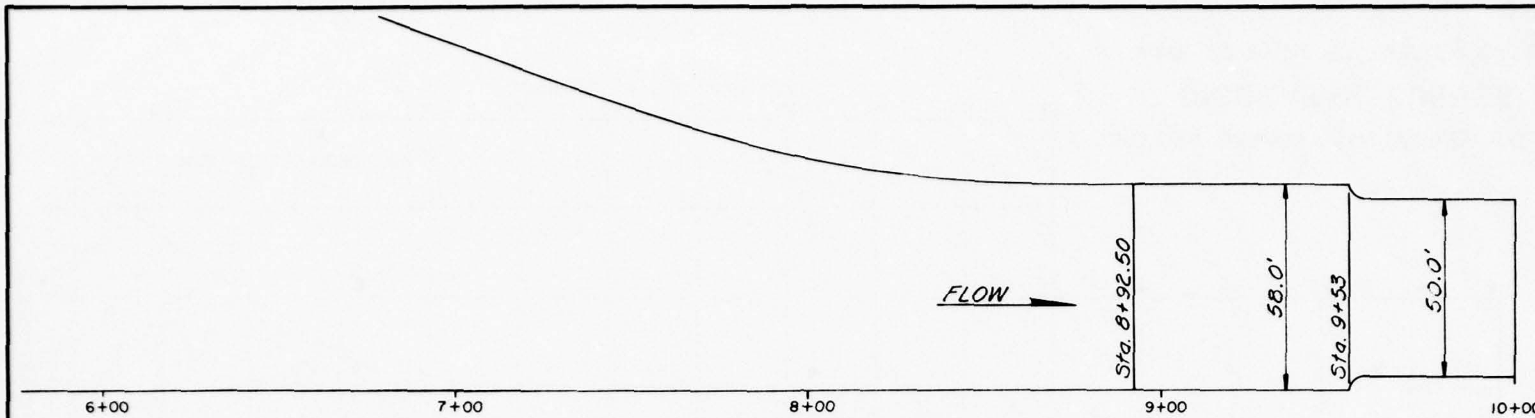
FINAL SPILLWAY DESIGN WITH ROCK PROTECTION TYPE 2



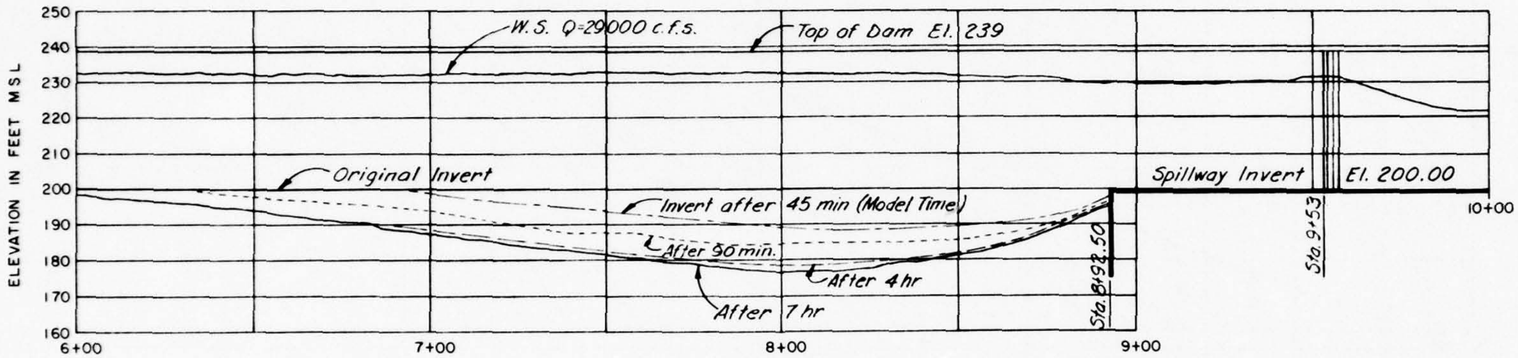
WHITTIER NARROWS-SAN GABRIEL SPILLWAY

DISCHARGE CURVES

NINE 50-BY 29-FT TAINTER GATES



PLAN OF SECTIONAL MODEL
ONE BAY



PROFILE

WHITTIER NARROWS-SAN GABRIEL SPILLWAY
APPROACH CHANNEL
(SINGLE BAY)
SCOUR UPSTREAM OF CONCRETE APRON
WITHOUT ROCK APRON

PART V: ROSEMEAD BLVD. DROP STRUCTURE

Original Design

57. The Rosemead Blvd. drop structure, built in the general model, was originally constructed as shown in Plate 40 and Photo 43. The concrete sloping headwall which had a drop height of 9 ft was located at the center line of Rosemead Blvd. Bridge. To minimize the velocity of flow, a 40-ft-long stilling basin with a 3-ft-high end sill was constructed. The design called for two rows of 2-ft-high baffle blocks located 13 and 23 ft downstream from the sloping headwall. A derrick stone apron extended 90 ft downstream from the end sill. Four bridge piers were aligned parallel to the basin center line. The paved invert upstream of the headwall under the bridge was placed at el 198.0. Hydraulically, this design was satisfactory but resulting structural and economical analysis proved unfavorable. Therefore it was desirable to pursue the design from an economical standpoint. Savings could possibly be realized by eliminating the stilling basin altogether.

Alternative Plans

58. Expedient alteration and easier construction resulted in the adoption of a 1:24-scale model in which experimentation was carried on to verify and approve theoretical data. Due to the symmetry of the Rosemead Blvd. Bridge about its center line, it was concluded that one half of the structure would be a suitable model to ascertain conclusive results. The transformation of the Rosemead drop structure resulted in the complete elimination of the stilling basin and replacement of it with a grouted cobblestone apron. The apron, beginning at the downstream edge of the bridge invert, has a slope of 1V on 2.5H. Plate 40 shows plan and profile of the alternative plans and rock protection.

Model construction

59. Bridge piers and abutment were constructed of plywood. Roughened concrete was used to simulate the grouted cobblestone invert

under the bridge and apron chute. The area upstream of the bridge was molded in sand. Rock in the stabilizing section downstream of the drop below the bridge was represented by 3/4-in. gravel. Sand was used to model the 450-ft channel downstream of the bridge. General views of the half-plan model are shown in Photo 44.

Sequence of tests

60. Table 12 lists all tests in order of the testing procedure with identifying description and illustrative references.

General procedure

61. The prototype discharges simulated during the tests were 40,000, 30,000, 20,000, and 10,000 cfs. Tests were concerned with overall flow conditions, energy dissipation characteristics, and depth and scour patterns in the channel immediately downstream from the drop structure. To obtain comparable scour of the sand bed for each test, all runs were of 1-hr duration. Data from all of the tests were supplemented by photographic records of the hydraulic action and scour patterns.

Tests 1-8 (rock plan 1)

62. The model was tested first with the bridge invert at el 198.0. Flow conditions were studied for discharges of 40,000, 30,000, 20,000, and 10,000 cfs with normal tailwater conditions. To substantiate visual observation, water-surface measurements were taken through the bridge and scour data were taken downstream from the drop structure for each specified discharge. The bridge invert was raised to el 199.0 and all above tests were repeated. The rock protection designated rock plan 1 in Plate 40 was used in the testing. Photographs of these tests are shown in Photos 45-52. Depth contours of the water surface through the bridge are shown in Plates 41-44. The scour patterns are shown in Plates 45-52. The 40,000-cfs discharge was the most erosive; however, the deepest part of the scour hole was 110 ft downstream of the bridge. The 30,000-cfs discharge caused more of the rock protection to be moved off the grouted-stone slope section of the drop structure. Scour profiles for all discharges with bridge inverts at el 198.0 and 199.0 are shown in Plate 53; velocity measurements for test 5 are shown in Plate 54.

Tests 9 and 10 (rock plan 1)

63. These two tests were made to determine the adequacy of the design under high tailwater conditions. Test 9 was a run of 40,000 cfs and a tailwater elevation at 211.5. Water surface was relatively smooth with the depth of scour much shallower than that in test 5. The scour pattern is shown in Plate 55. Photographs for test 9 are shown in Photo 53. In test 10 the discharge was 30,000 cfs with a tailwater elevation set at 211.0. The general results were similar to those in test 9. The flow was stable and well distributed, resulting in smoother water surface and shallower depth of scour than that in test 6. Scour pattern is shown in Plate 56 and photographs in Photo 54.

Test 11 (rock plan 1)

64. For a tailwater elevation of 208.0 and a discharge of 30,000 cfs, more waves were transmitted downstream from the drop structure. The undulating flow conditions and the ensuing scour are shown in Photo 55; see Plate 57 for scour pattern.

Test 12 (rock plan 2)

65. This test was the same as test 11 except that rock protection plan 2 (Plate 40) was used in the testing. The rock protection forms a 40-ft stilling basin with a 5.5-ft-high sloping end sill. The rock apron (basin) is 11 ft below the bridge invert. Photo 56 shows the rock stilling basin before the run. Photographs of the hydraulic action were not taken. Scour photographs are shown in Photo 57. Scour pattern for this test is shown in Plate 58. The rock apron provided adequate protection downstream from the drop structure.

Tests 13 and 14 (rock plan 2)

66. Tests 13 and 14 are the same as tests 5 and 6, respectively, except for the rock protection. Runs of 40,000 and 30,000 cfs were made with normal tailwater conditions. These tests produced a favorable hydraulic action. Test 13 developed a series of rollers and waves downstream. In test 14, the jump was good and the water surface was smooth in the downstream channel. Photos 58 and 59 show the hydraulic performance and scour. Scour patterns for runs of 40,000 and 30,000 cfs are shown in Plates 59 and 60, respectively. Results were generally similar

to those obtained in tests 5 and 6; however, in tests 13 and 14 the deepest parts of the scour holes were much farther downstream because of rock plan 2. As expected, the run of 40,000 cfs produced the maximum water-surface elevation of 210.0 at the upstream end of the bridge. The average water-surface elevation through the bridge was 209.0. The elevation of low steel was 210.1, 4 ft upstream from the center line of the bridge and 210.7 downstream from the center line of the bridge. Tops of piers are at el 210.0.

67. In all tests, the rock protection never scoured below the simulated grouted-stone slope section of the drop. Local displacement of the rock protection occurred but the amount of rock transported downstream was very small.

Application of Final Design of Drop Structure to General Model

68. Following the half-scale model test, the final design of the drop structure was tested in the general 1:60-scale model. Scour downstream from the grouted-stone drop structure was observed for combination discharges varying between 10,000 and 30,000 cfs from the San Gabriel River and between 0 and 10,000 cfs from the Rio Hondo. To obtain comparable scours of the sand bed between the general model and the half-scale model, all runs were of 38-min (model time) duration which is equivalent to the 1-hr duration used in the 1:24-scale model.

69. The combination of 30,000 cfs from the San Gabriel River and no flow in the Rio Hondo produced the greatest scour because of the low tailwater condition on the downstream side of the drop structure. The lowest point of scour was el 176.5. Scour photographs are shown in Photo 60. With a combination of 30,000 cfs from the San Gabriel River and 10,000 cfs flowing in Rio Hondo and tailwater elevation set at el 208.0, the scour downstream of the drop structure was not excessive (Photo 61). Photo 62 shows the results of scour with the same combination, but with normal tailwater conditions (el 203.5). The lowest point of scour was el 185.0.

70. In view of the favorable results in the half-scale model and the general model, this drop structure was recommended for the prototype.

Table 12

Sequence of Tests on Rosemead Blvd. Drop Structure

Tests	Discharge cfs	Tailwater Elevation	Bridge Invert Elevation	Plate References Depth Contours	Plate References Scour Pattern	Photo References Flow and/or Scour	Rock Plan
1	40,000	207.7	198.0	41	45	45	1
2	30,000	203.5	198.0	42	46	46	1
3	20,000	198.9	198.0	41	47	48	1
4	10,000	197.1	198.0	42	48	48	1
5	40,000	207.7	199.0	43	49	49	1
6	30,000	203.5	199.0	43	50	50	1
7	20,000	198.9	199.0	44	51	51	1
8	10,000	197.1	199.0	44	52	52	1
9	40,000	211.5	199.0	--	55	53	1
10	30,000	211.0	199.0	--	56	54	1
11	30,000	208.0	199.0	--	57	55	1
12	30,000	208.0	199.0	--	58	57	2
13	40,000	207.7	199.0	--	59	58	2
14	30,000	203.5	199.0	--	60	59	2

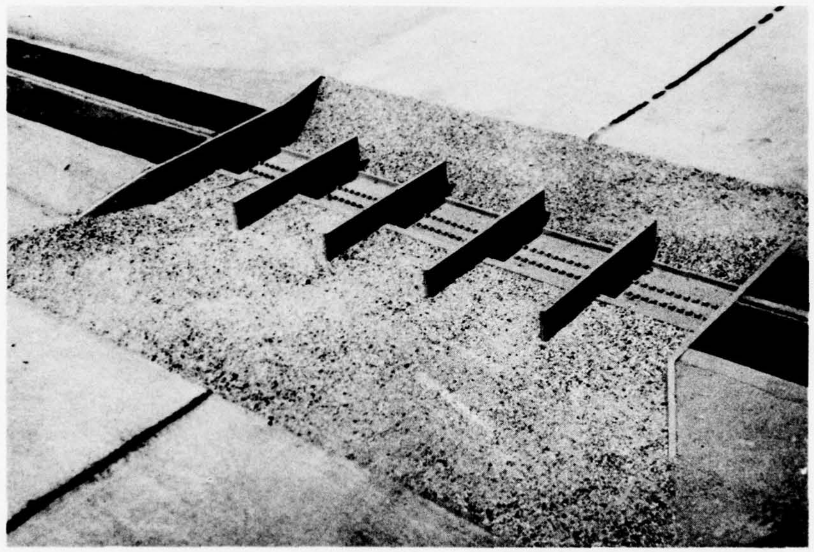
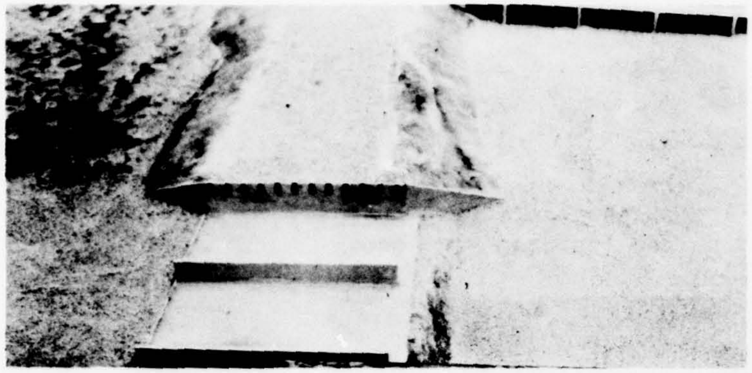
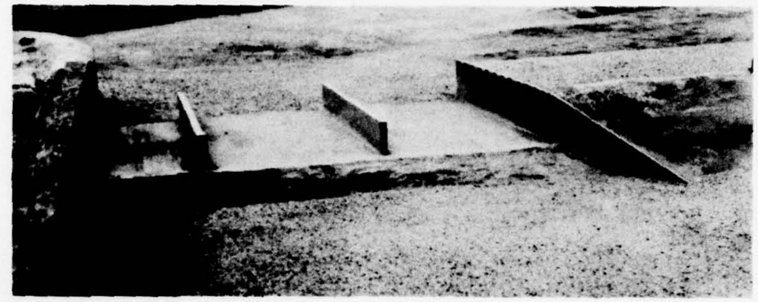


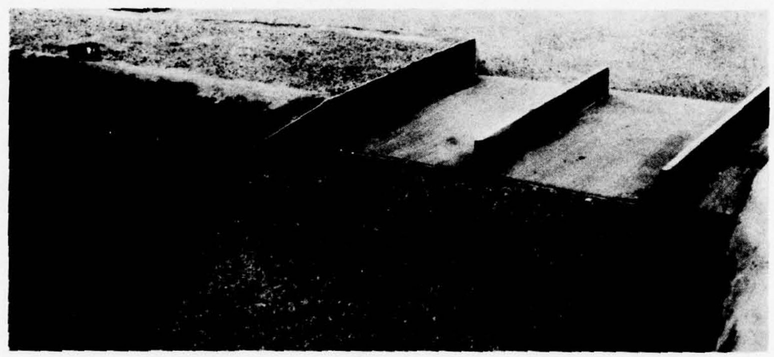
Photo 43. Original design 1:60-scale model, Rosemead Blvd. Bridge drop structure. Flow from left to right



Flow left to right

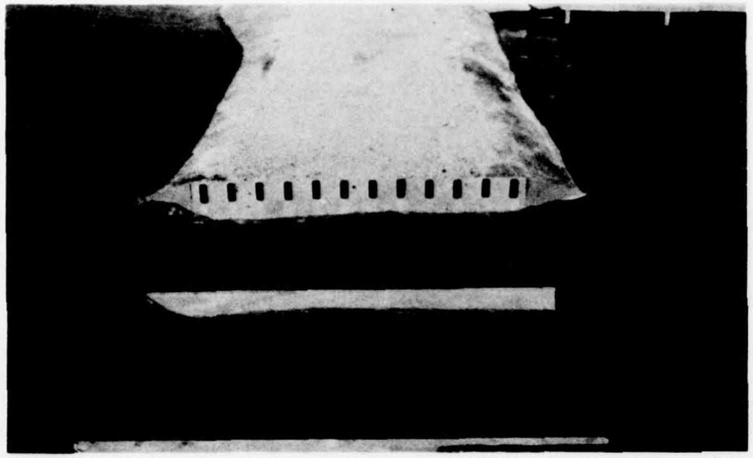


Looking upstream

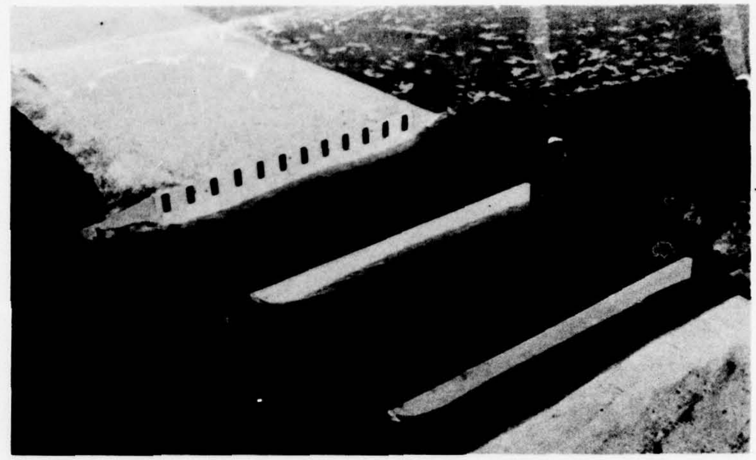


Looking downstream

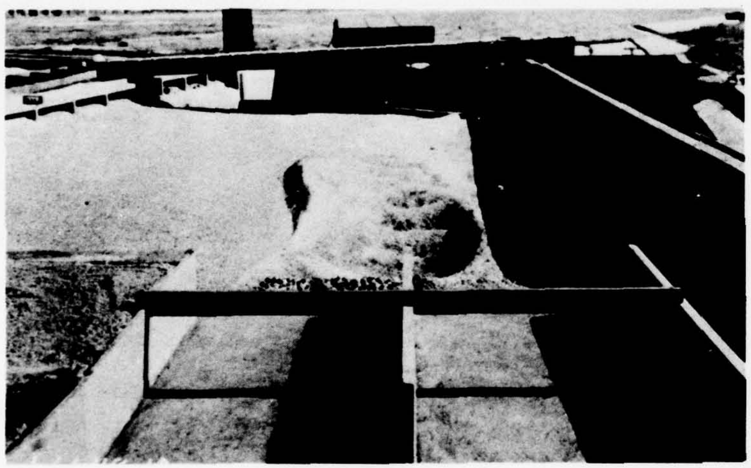
Photo 44. Alternative design 1:24 half-plan model, Rosemead Blvd. Bridge drop structure. The black rectangles on bridge abutment represent end of beams



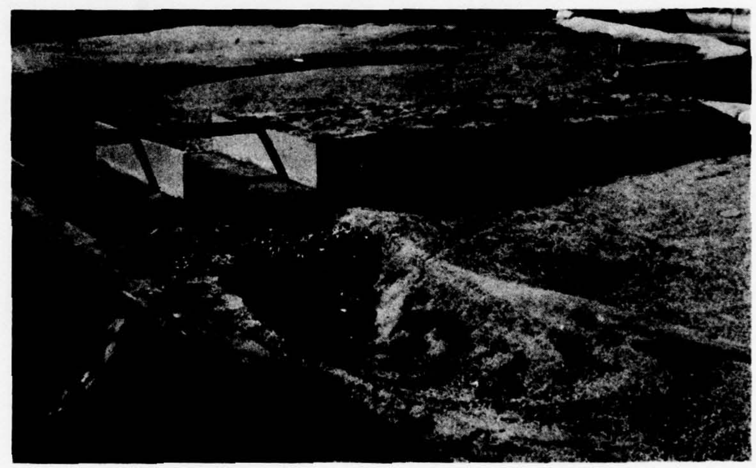
Flow left to right



Looking downstream from right side

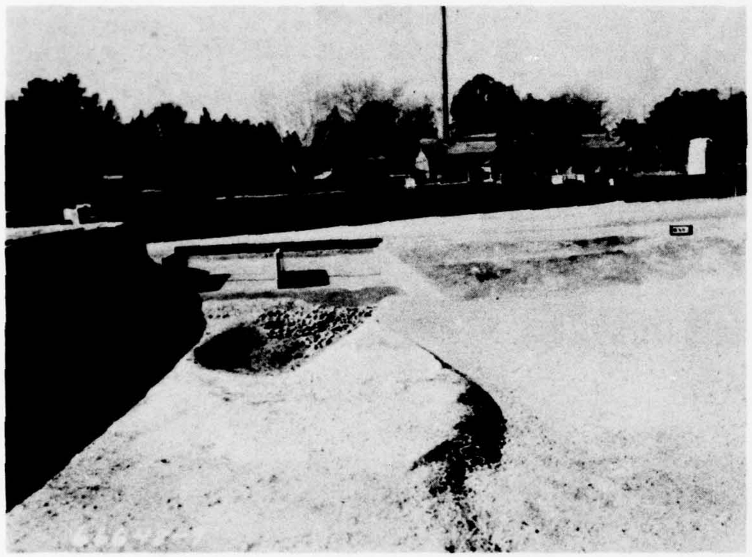


Scour after 1-hr run, looking downstream



Scour after 1-hr run, looking upstream

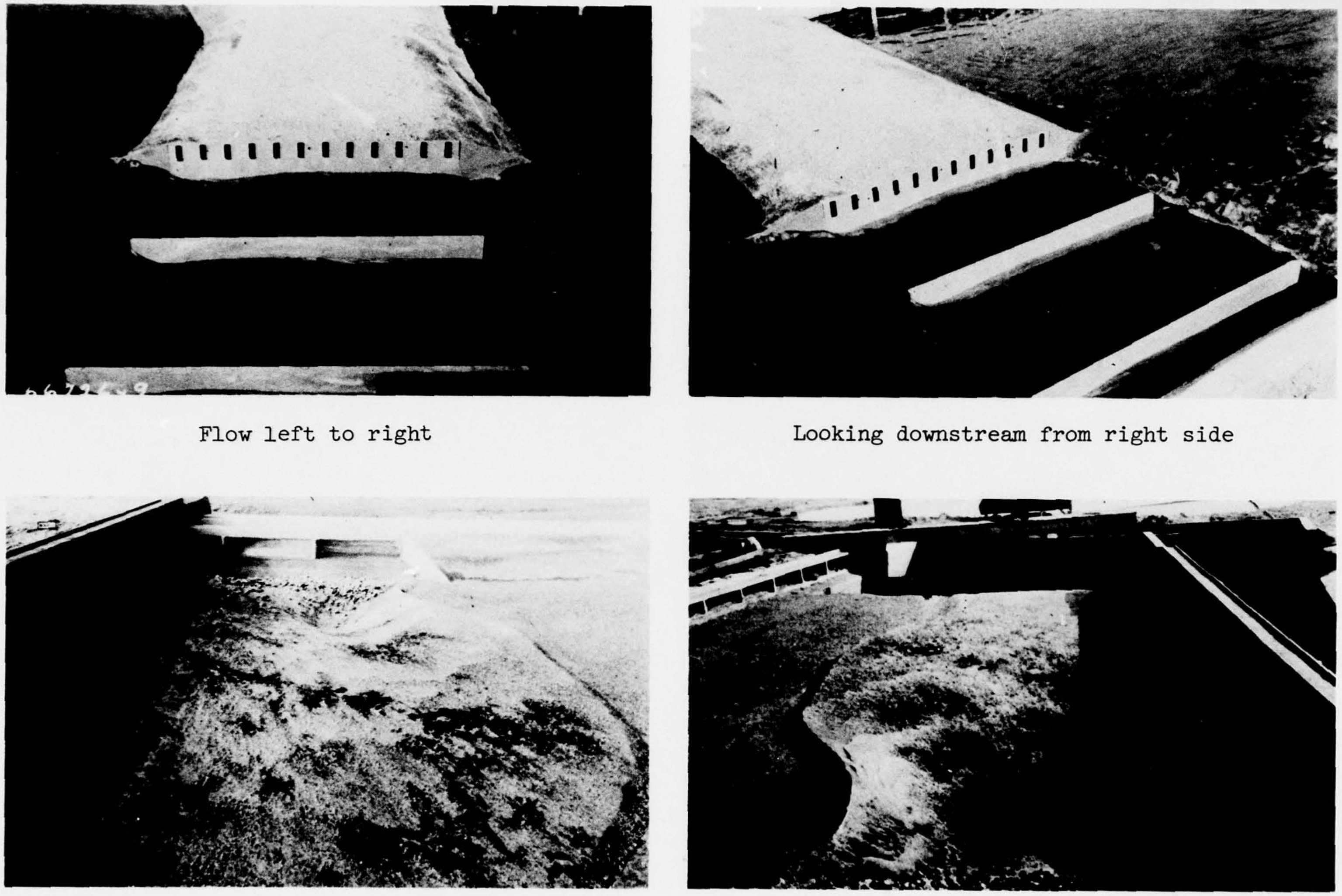
Photo 45. Test 1; discharge 40,000 cfs, tailwater el 207.7, bridge invert el 198.0 (sheet 1 of 2)



Looking upstream

looking upstream from
right side

Photo 45 (sheet 2 of 2)



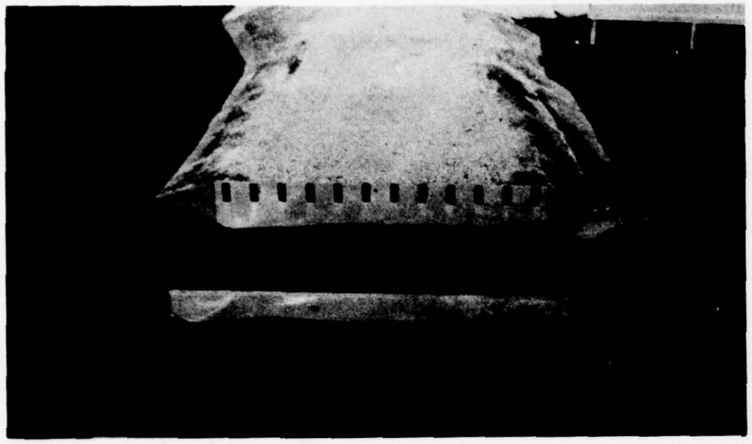
Flow left to right

Looking downstream from right side

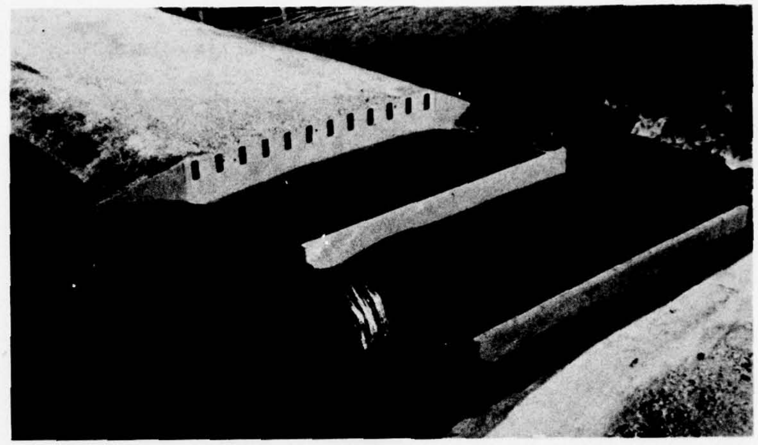
Scour after 1-hr run, looking upstream

Scour after 1-hr run, looking downstream

Photo 46. Test 2; discharge 30,000 cfs, tailwater el 203.5, bridge invert el 198.0

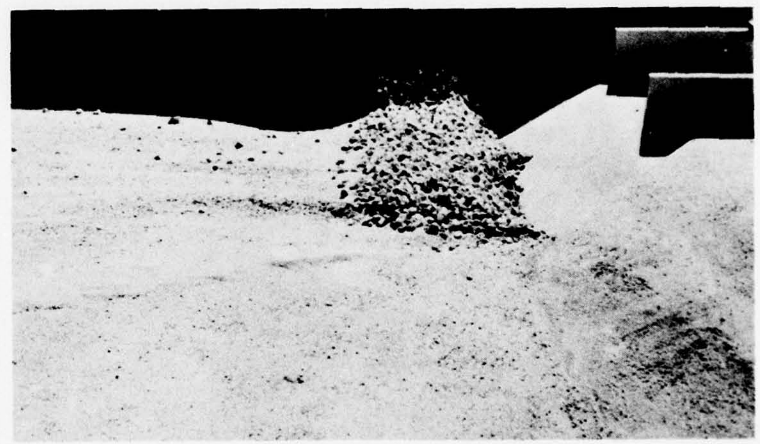


Flow left to right



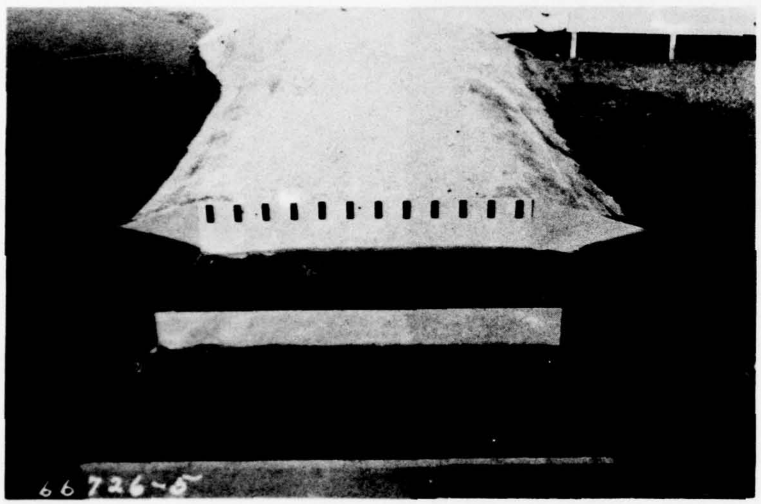
Looking downstream from right side

Scour after 1-hr run, looking upstream

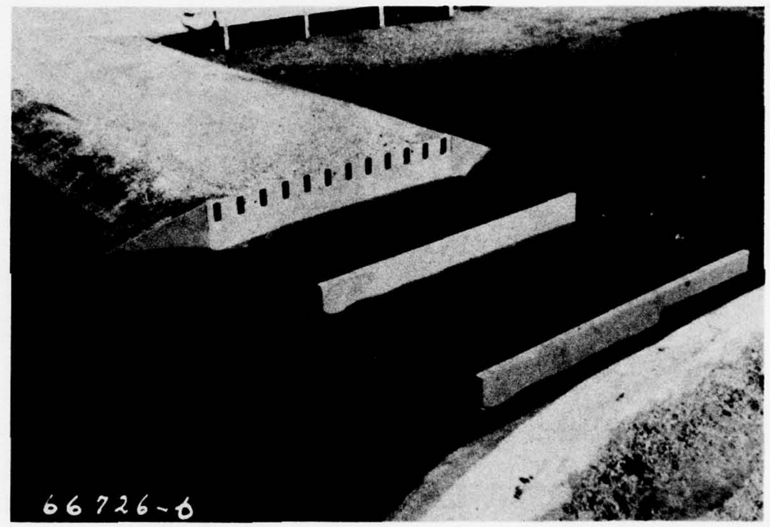


Scour after 1-hr run, looking across channel from left side

Photo 47. Test 3; discharge 20,000 cfs, tailwater el 198.9, bridge invert el 198.0

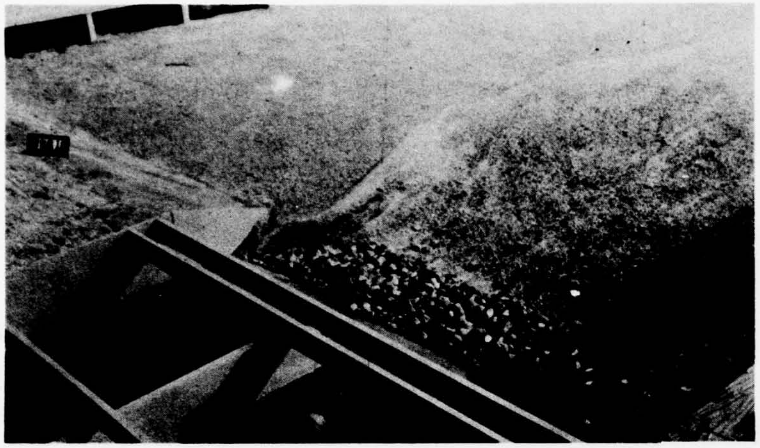


Flow left to right

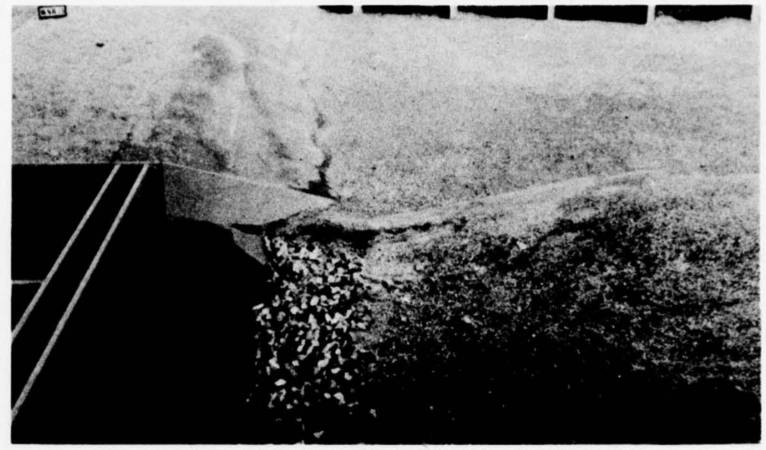


Looking downstream from right side

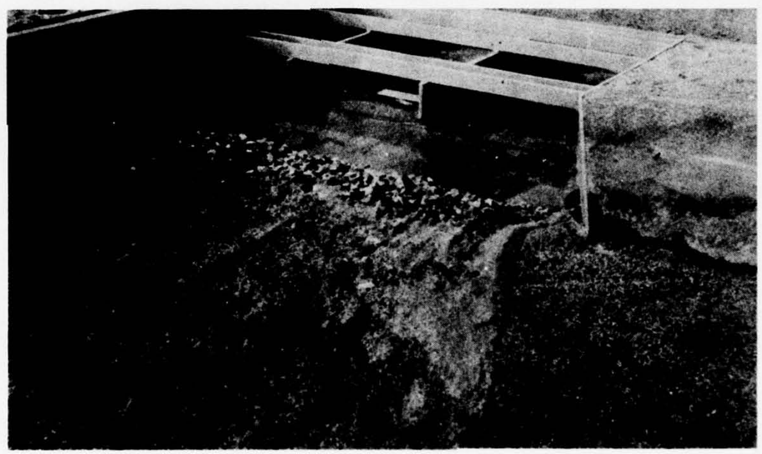
Photo 48. Test 4; discharge 10,000 cfs, tailwater el 197.1, bridge invert el 198.0 (sheet 1 of 2)



Looking downstream from right side



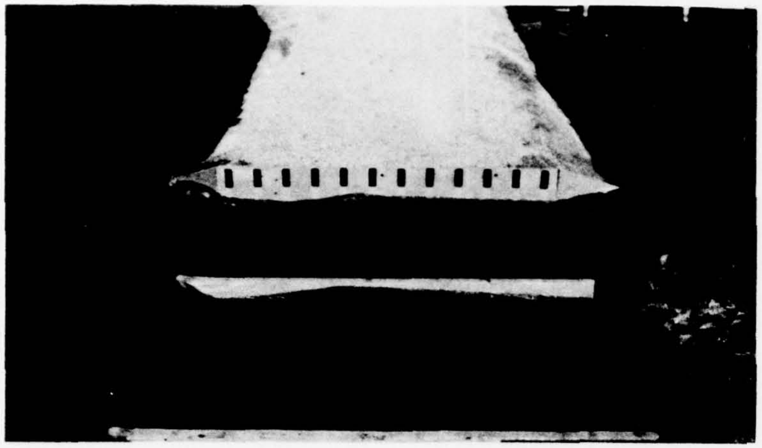
Looking across channel from right side



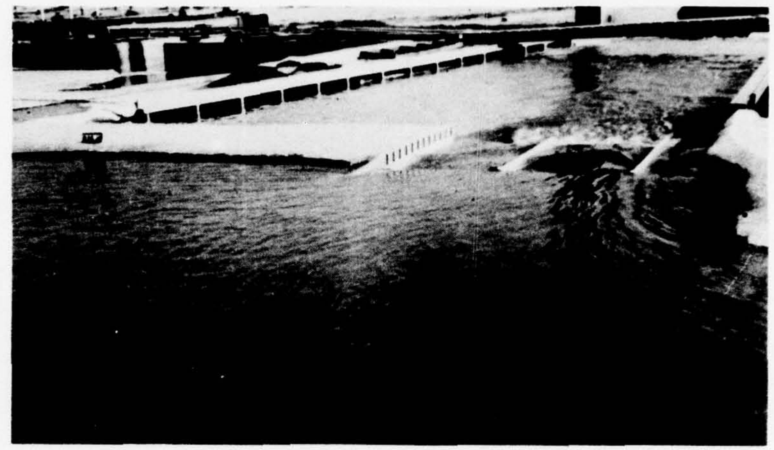
Looking upstream from left side

Scour after 1-hr run

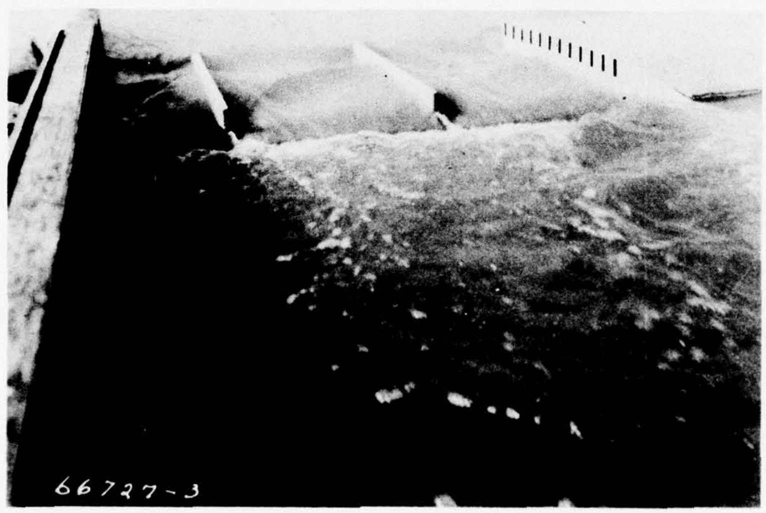
Photo 48 (sheet 2 of 2)



Flow left to right



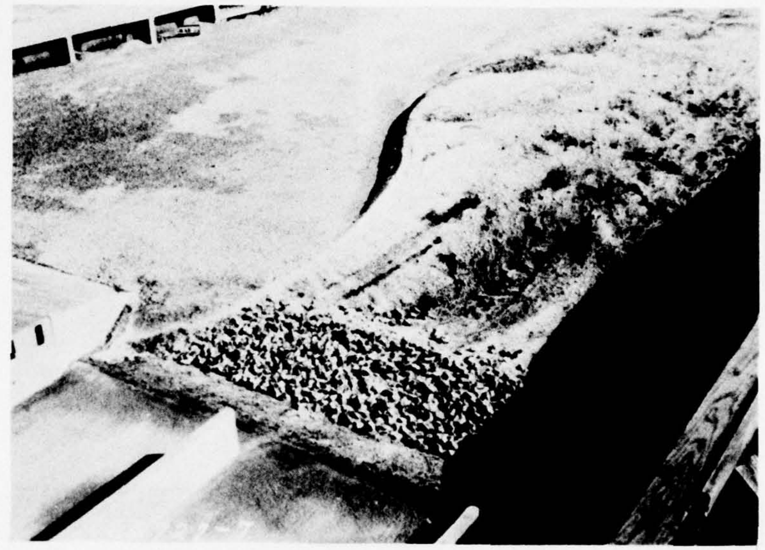
Looking downstream



Looking upstream

Photo 49. Test 5; discharge 40,000 cfs, tailwater el 207.7, bridge invert el 199.0 (sheet 1 of 2)

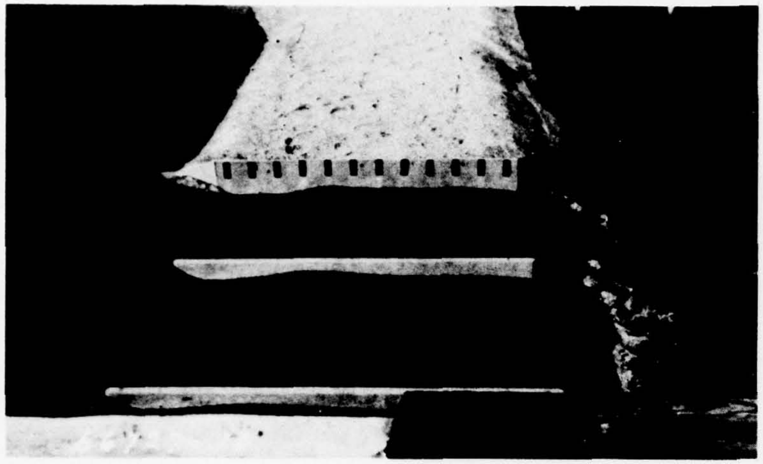
Looking upstream

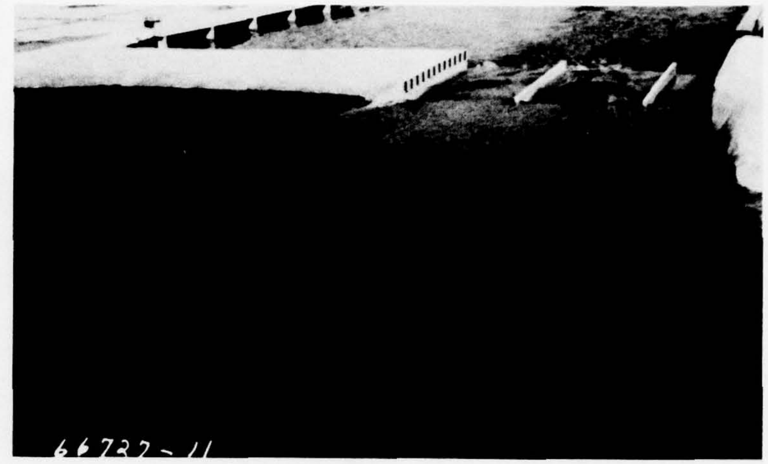


Looking downstream from right side

Scour after 1-hr run

Photo 49 (sheet 2 of 2)


Flow left to right



Looking downstream

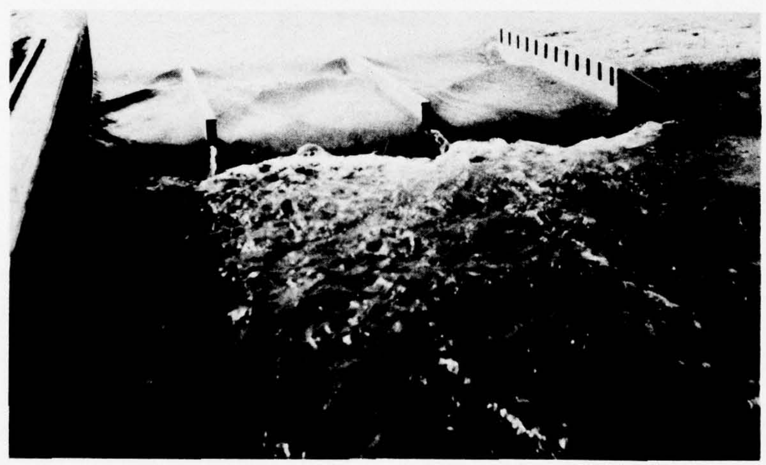
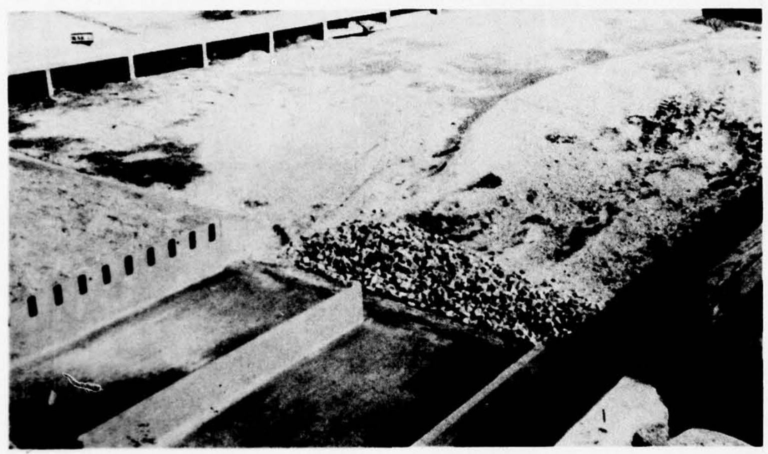

Looking upstream

Photo 50. Test 6; discharge 30,000 cfs, tailwater el 203.5, bridge invert el 199.0 (sheet 1 of 2)

Looking upstream

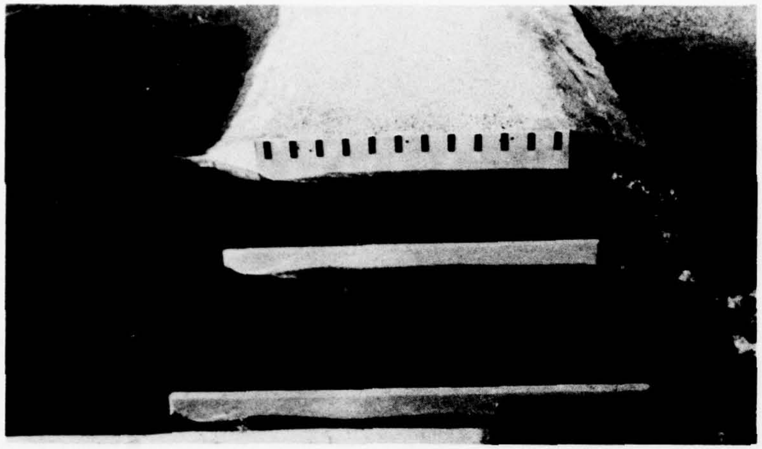
Looking across channel from left side



Looking downstream from right side

Scour after 1-hr run

Photo 50 (sheet 2 of 2)



Flow left to right

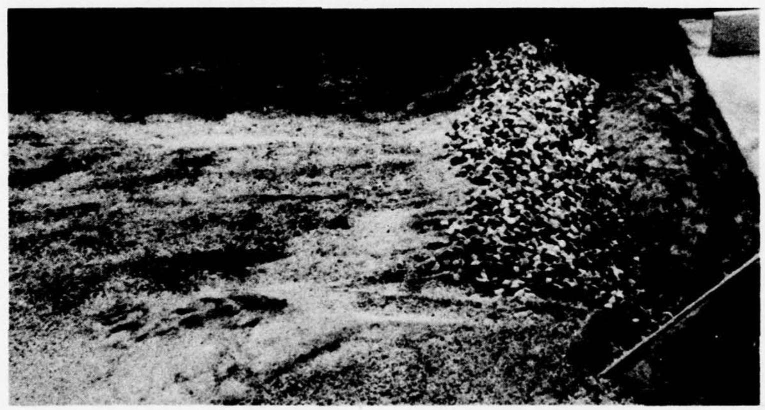
Looking downstream

Looking upstream

Photo 51. Test 7; discharge 20,000 cfs, tailwater el 198.9, bridge invert el 199.0 (sheet 1 of 2)

Looking upstream

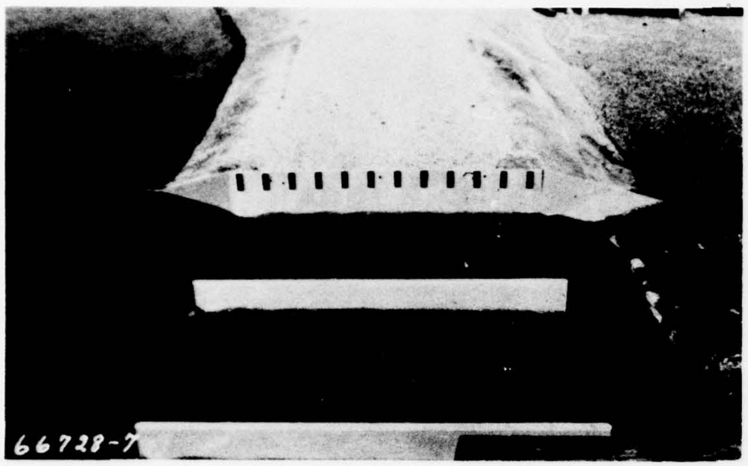
Looking downstream from right side



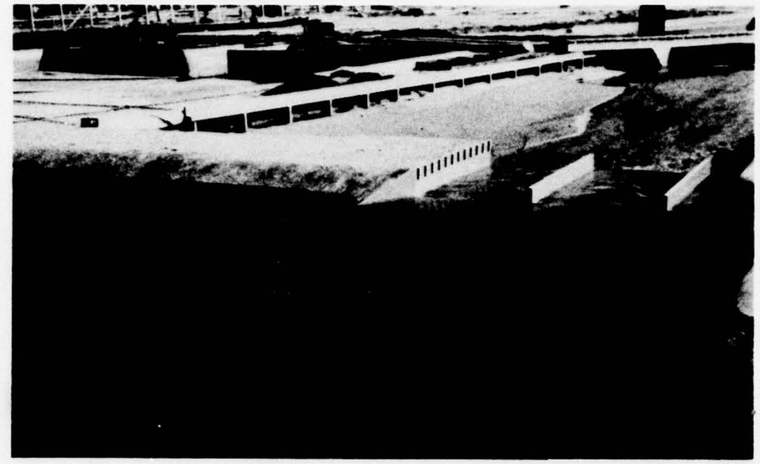
Looking across channel from left side

Scour after 1-hr run

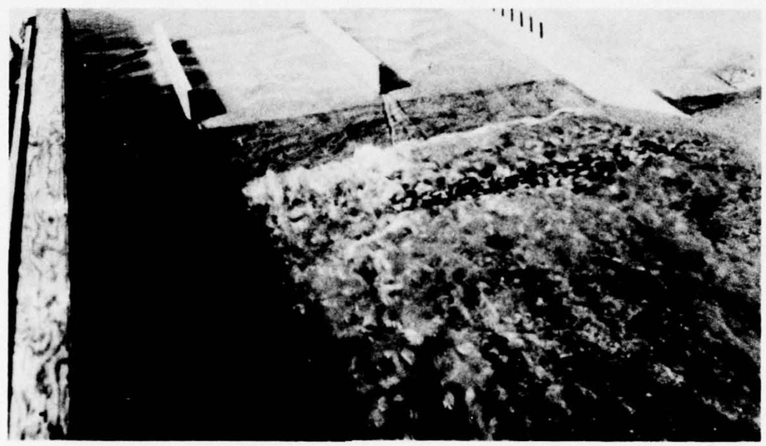
Photo 51 (sheet 2 of 2)



Flow left to right

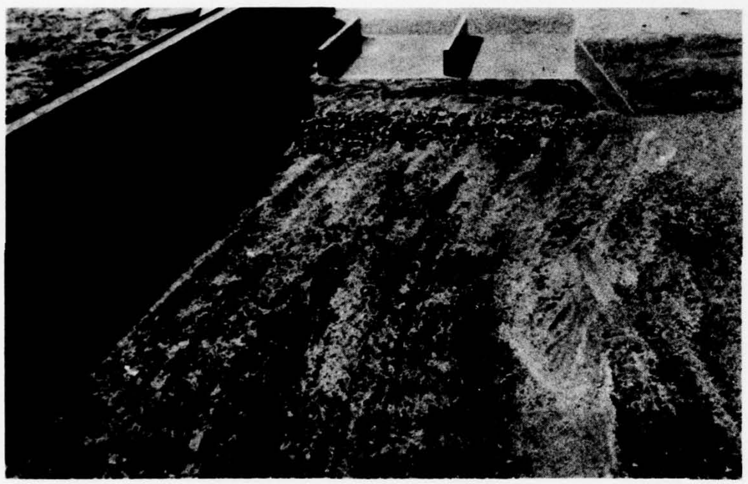


Looking downstream

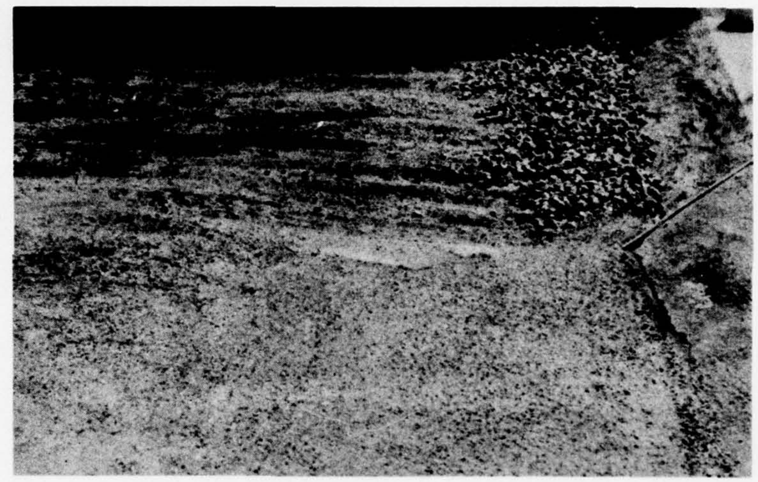


Looking upstream

Photo 52. Test 8; discharge 10,000 cfs, tailwater el 197.1, bridge invert el 199.0 (sheet 1 of 2)



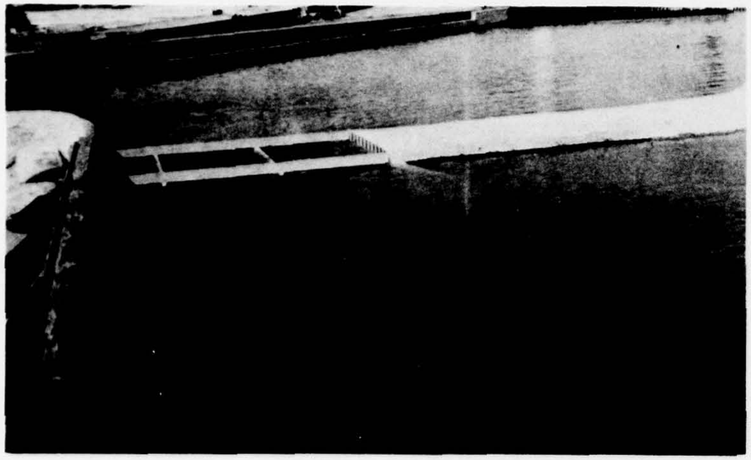
Looking upstream



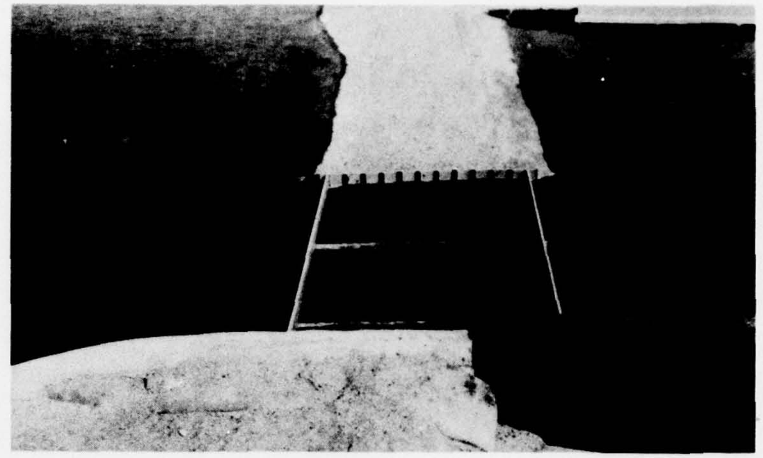
Looking across channel from left side

Scour after 1-hr run

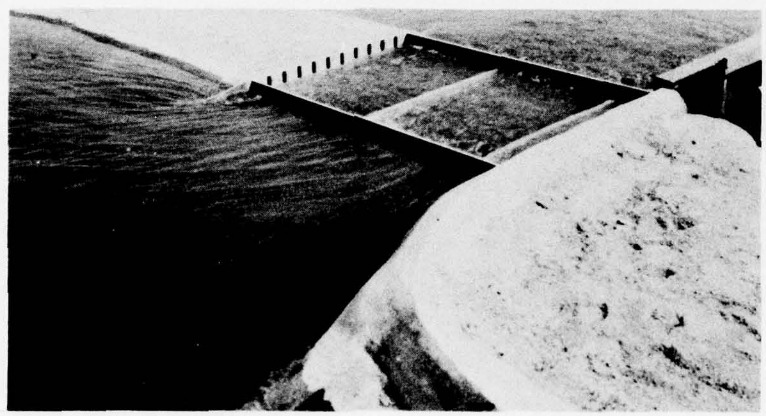
Photo 52 (sheet 2 of 2)



Looking upstream

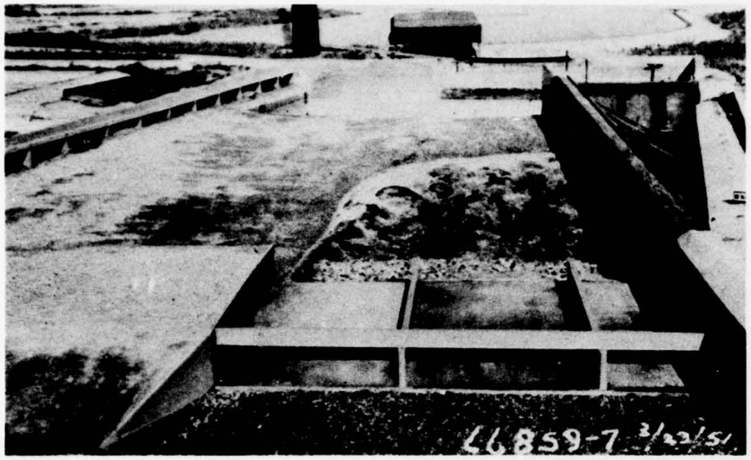


Flow left to right

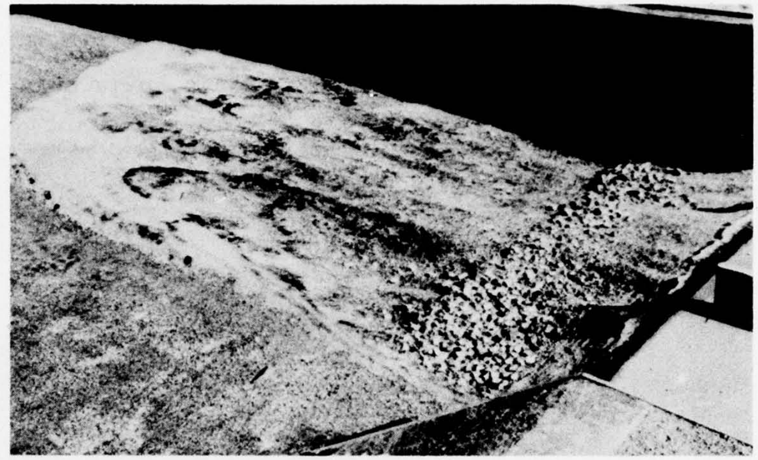


Looking downstream from right side

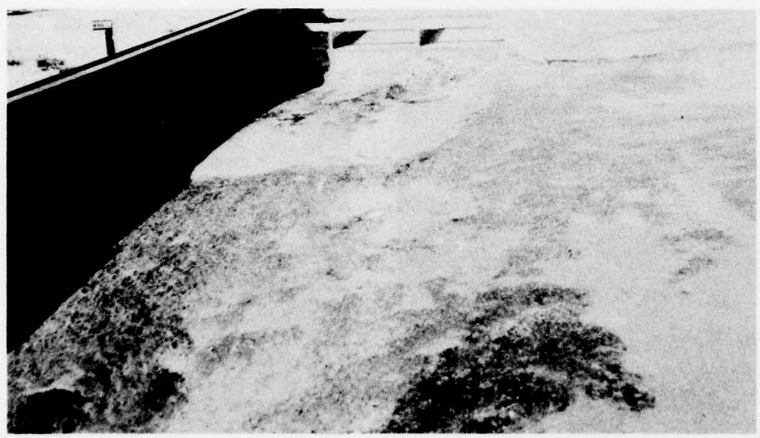
Photo 53. Test 9; discharge 40,000 cfs, tailwater el 211.5, bridge invert el 199.0 (sheet 1 of 2)



Looking downstream



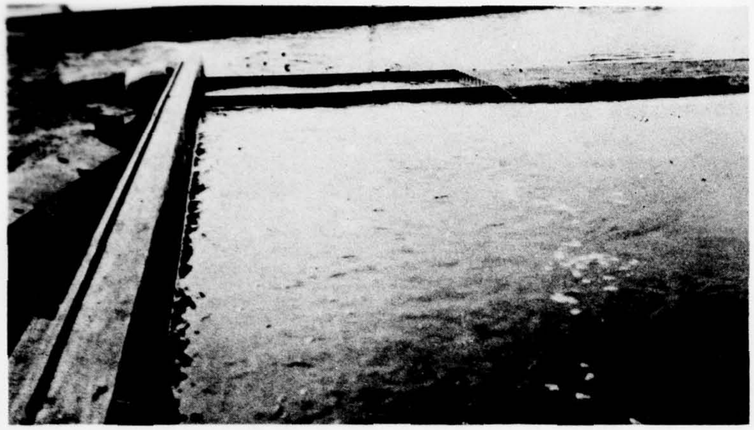
Looking downstream from left side



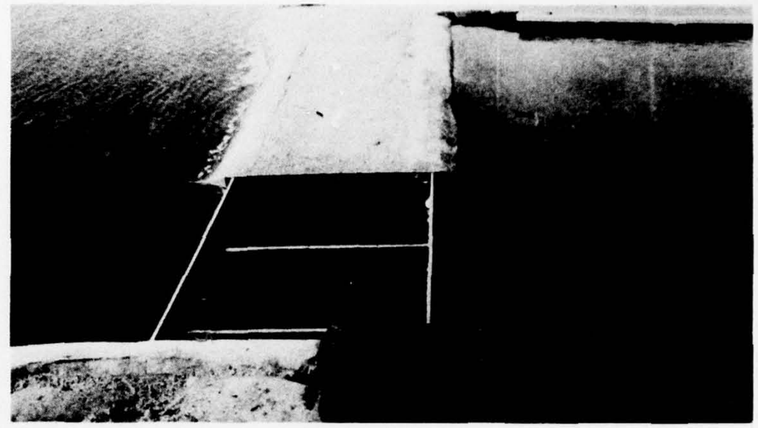
Looking upstream

Scour after 1-hr run

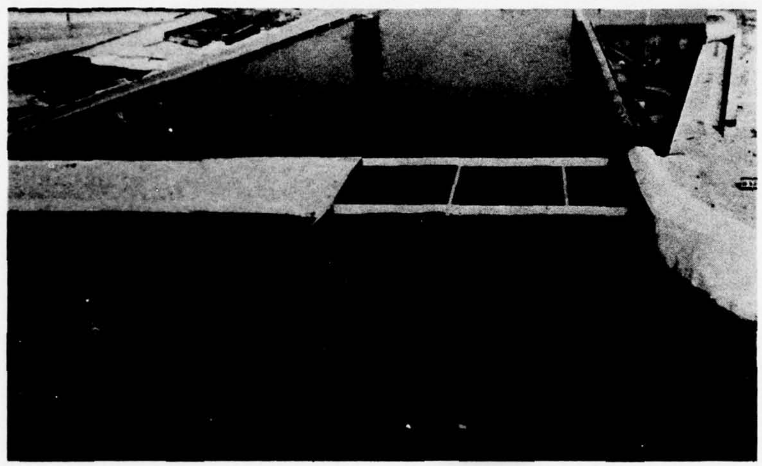
Photo 53 (sheet 2 of 2)



Looking upstream

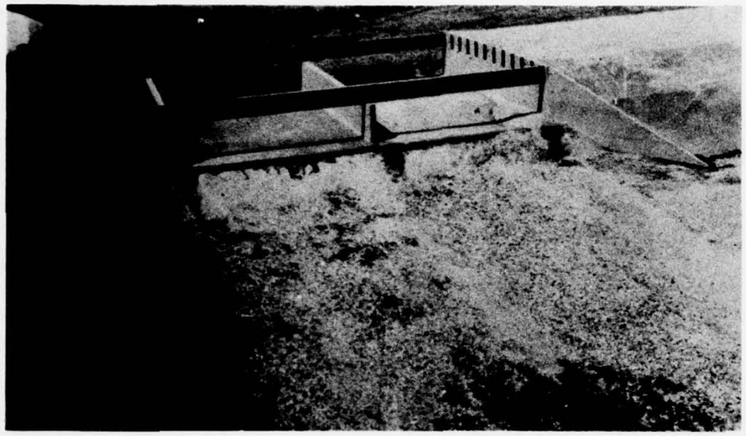


Flow left to right

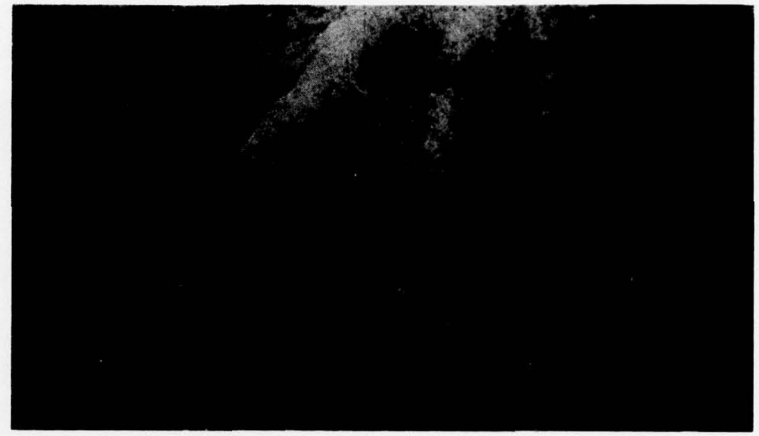


Looking downstream

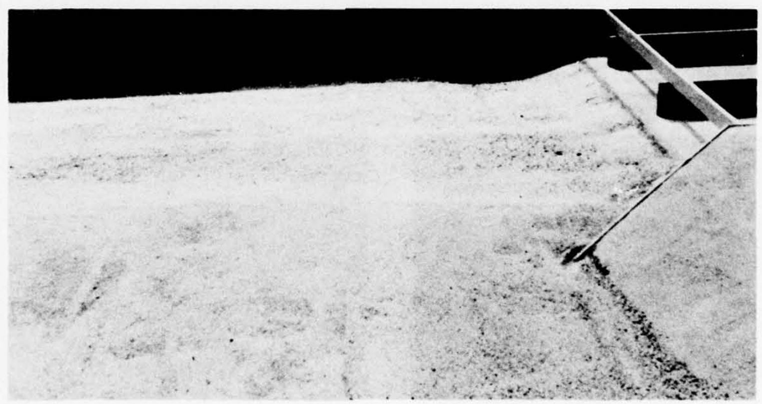
Photo 54. Test 10; discharge 30,000 cfs, tailwater el 211.0, bridge invert el 199.0 (sheet 1 of 2)



Looking upstream



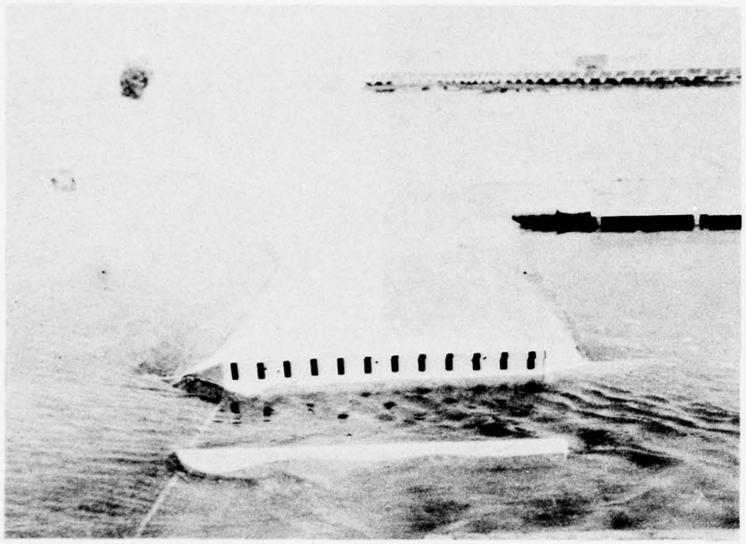
Looking downstream



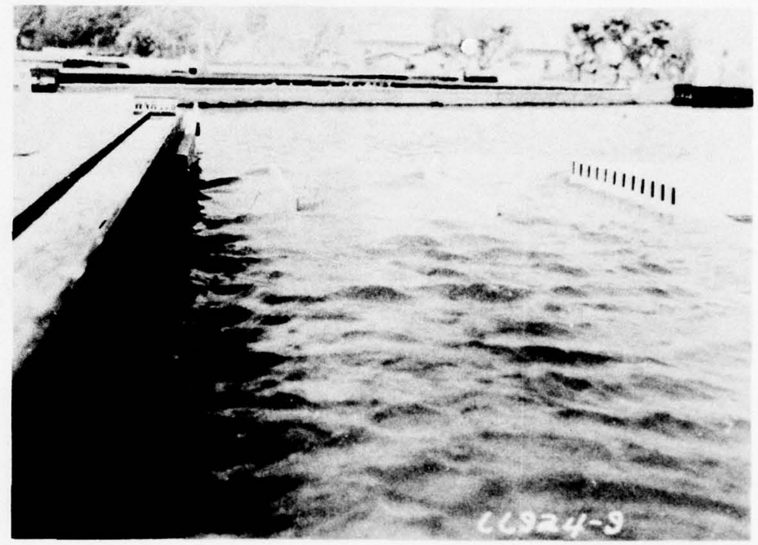
Looking across channel from left side

Scour after 1-hr run

Photo 54 (sheet 2 of 2)

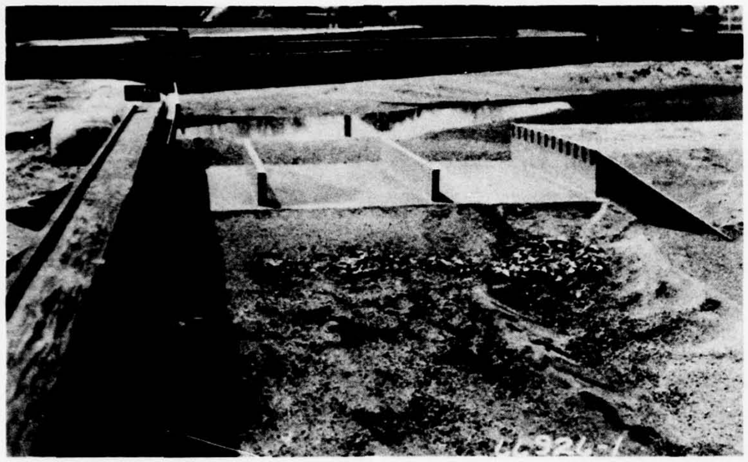


Flow left to right

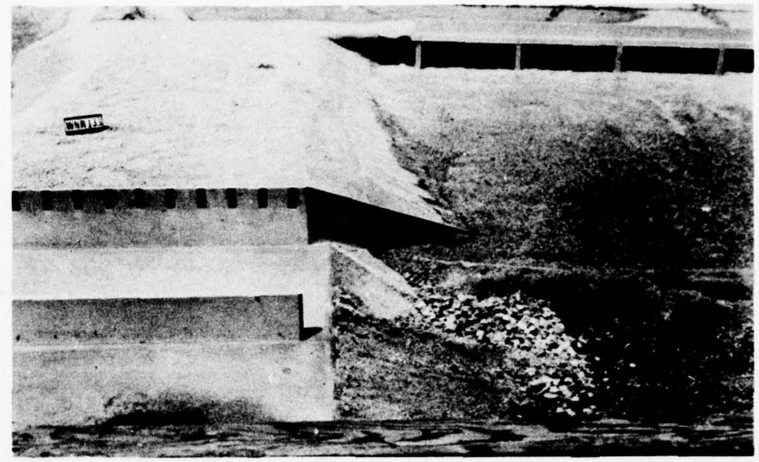


Looking upstream

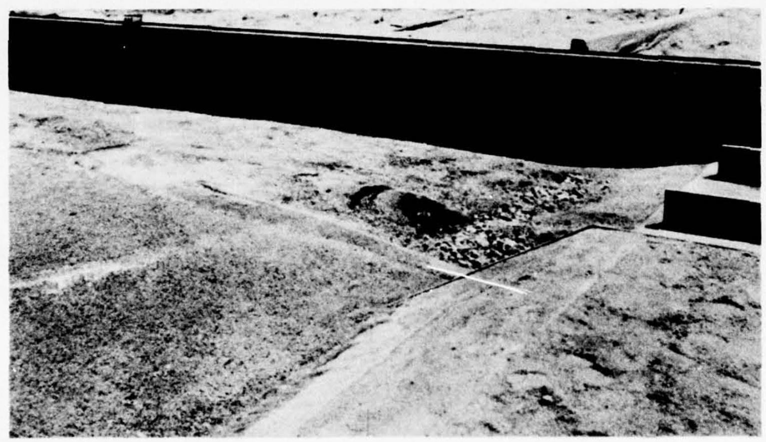
Photo 55. Test 11: discharge 30,000 cfs, tailwater el 208.0, bridge invert el 199.0 (sheet 1 of 2)



Looking upstream



Looking across channel from right side

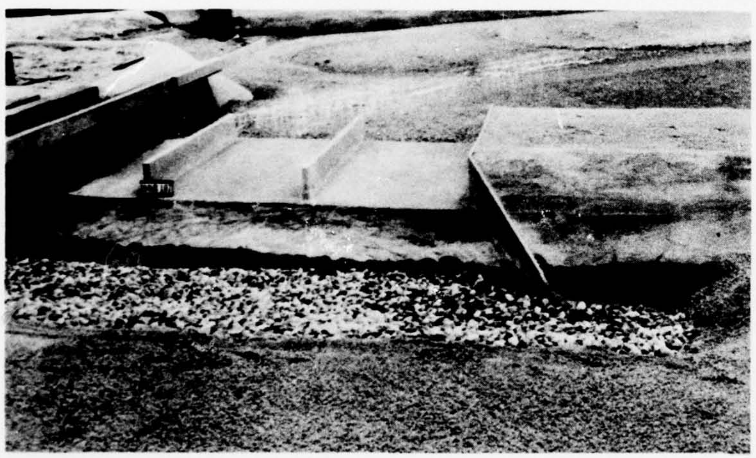


Looking downstream from left side

Scour after 1-hr run

Photo 55 (sheet 2 of 2)

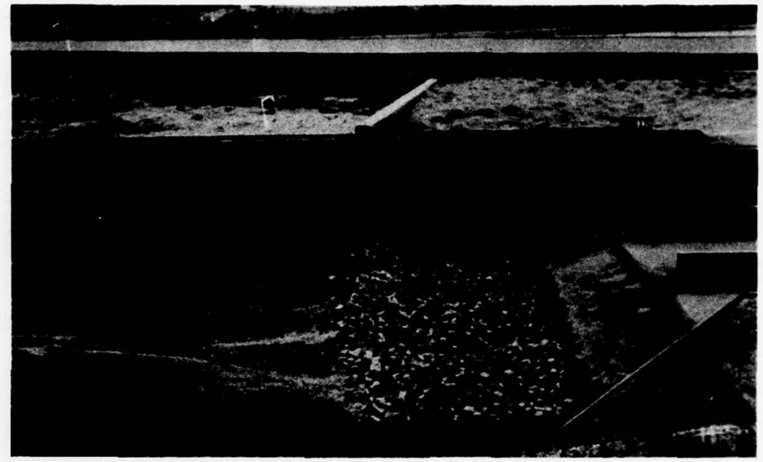
Looking across channel from left side



Looking upstream

Photo 56. Rock protection plan 2

Looking downstream



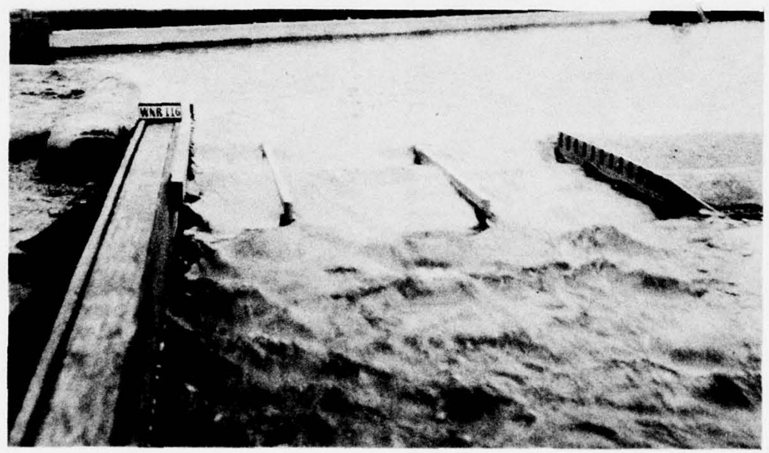
Looking across channel from left side

Looking downstream from left side

Photo 57. Test 12, scour after 1-hr run; discharge 30,000 cfs, tailwater el 208.0, bridge invert el 199.0

Flow left to right

Looking across channel from left side



Looking upstream

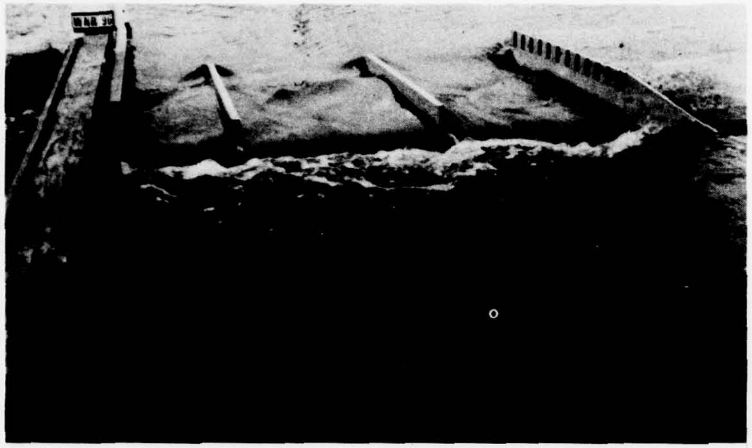
Photo 58. Test 13; discharge 40,000 cfs, tailwater el 207.7, bridge invert el 199.0 (sheet 1 of 2)

Looking upstream

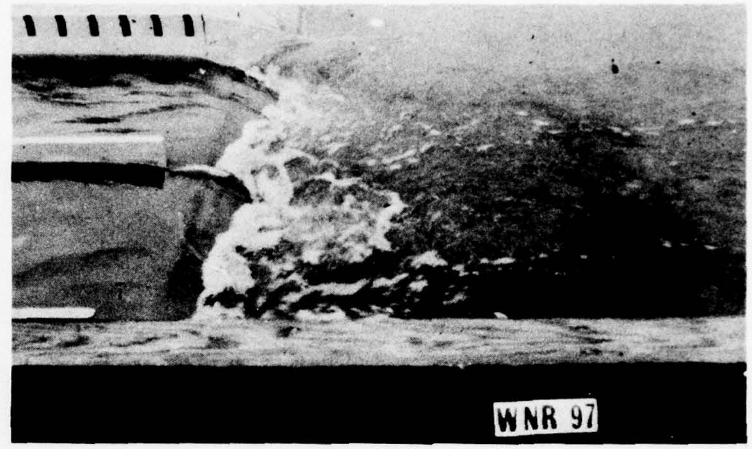
Looking across channel from left side

Scour after 1-hr run

Photo 58 (sheet 2 of 2)



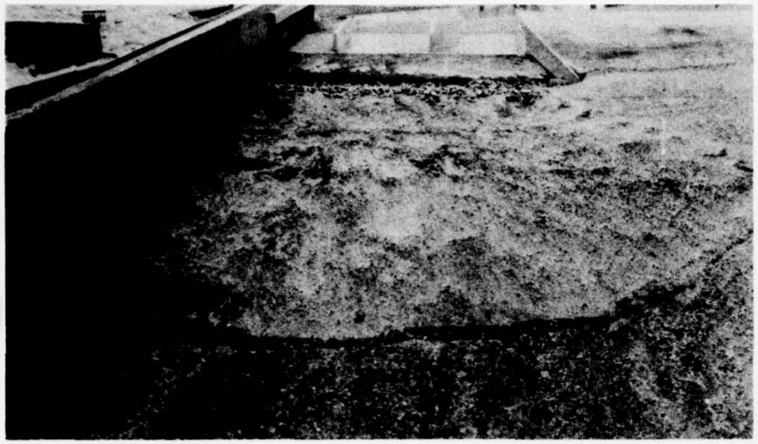
Looking upstream



Flow left to right

Looking across channel from left side

Photo 59. Test 14; discharge 30,000 cfs, tailwater el 203.5, bridge invert el 199.0 (sheet 1 of 2)



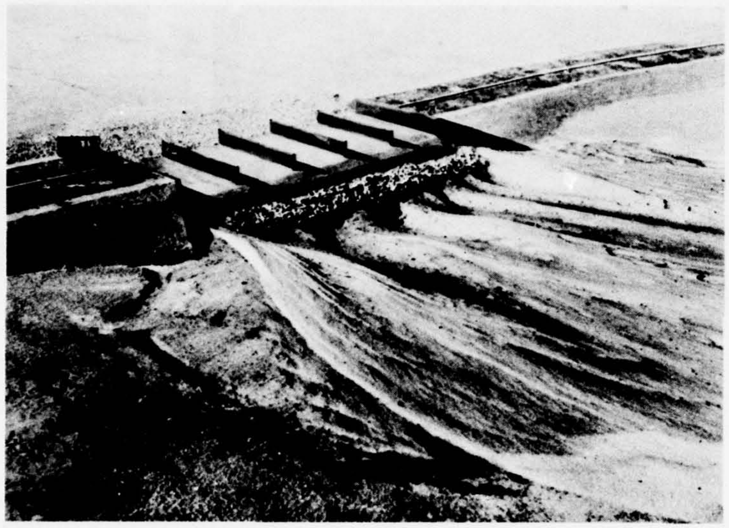
Looking upstream

Looking across channel from left side

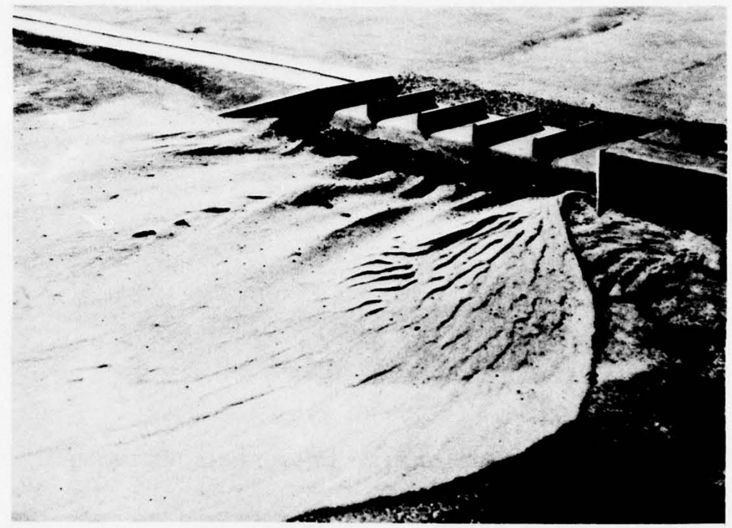
Looking upstream from left side

Scour after 1-hr run

Photo 59 (sheet 2 of 2)

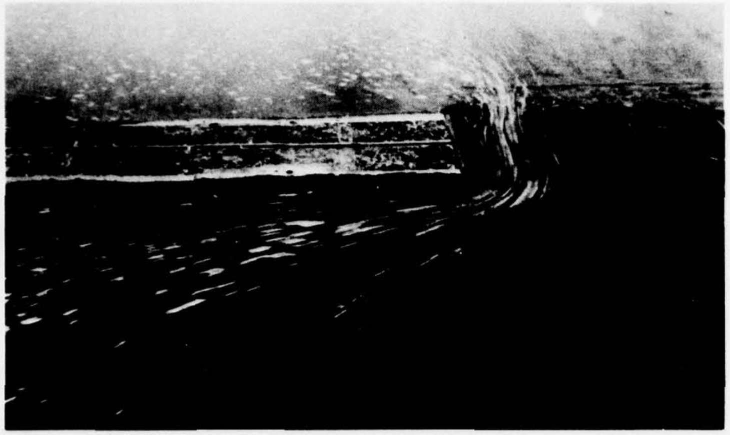


Looking upstream from right side

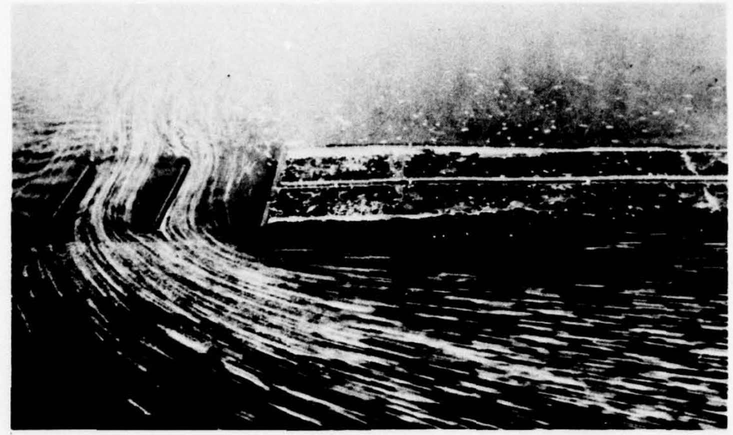


Looking upstream from left side

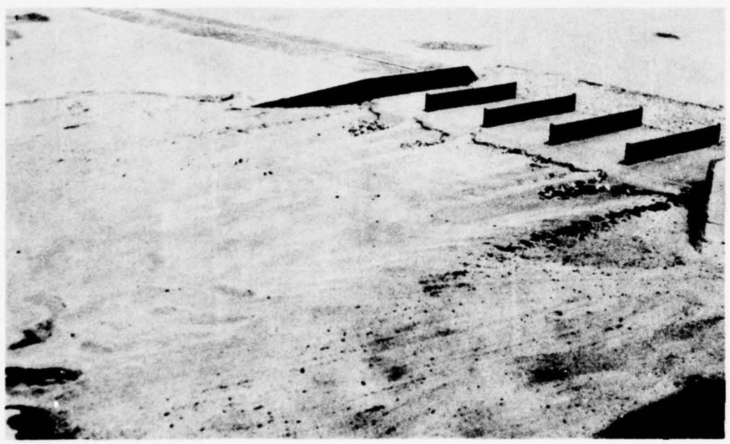
Photo 60. General model Rosemead drop structure, scale 1:60. Scour downstream of bridge after 38-min run (model); discharge 30,000 cfs through bridge and none in Rio Hondo, low tailwater conditions



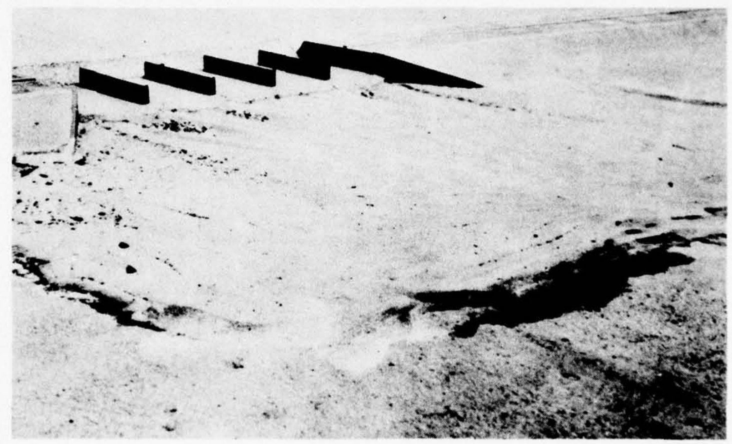
Looking downstream, left half of bridge



Looking downstream, right half of bridge



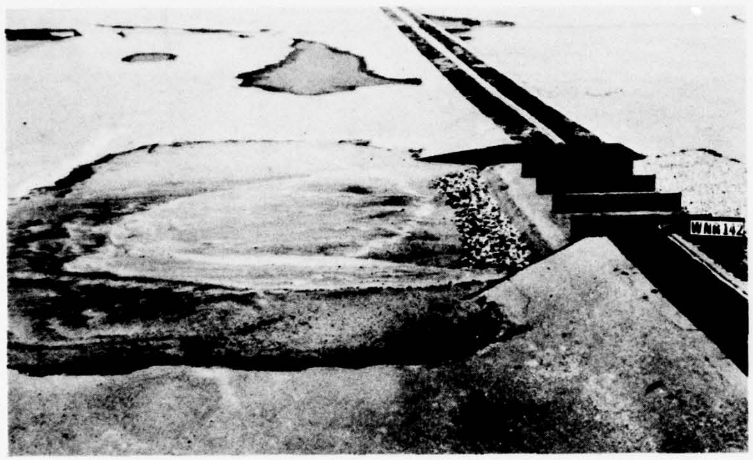
Looking upstream from left side



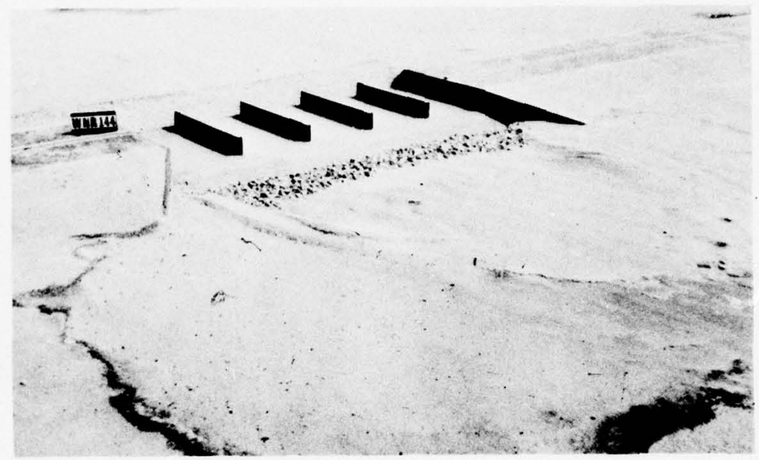
Looking upstream from right side

Scour downstream of bridge after 38-min run (model)

Photo 61. General model Rosemead drop structure, scale 1:60; discharge 30,000 cfs through bridge and 10,000 cfs in Rio Hondo, tailwater el 208.0

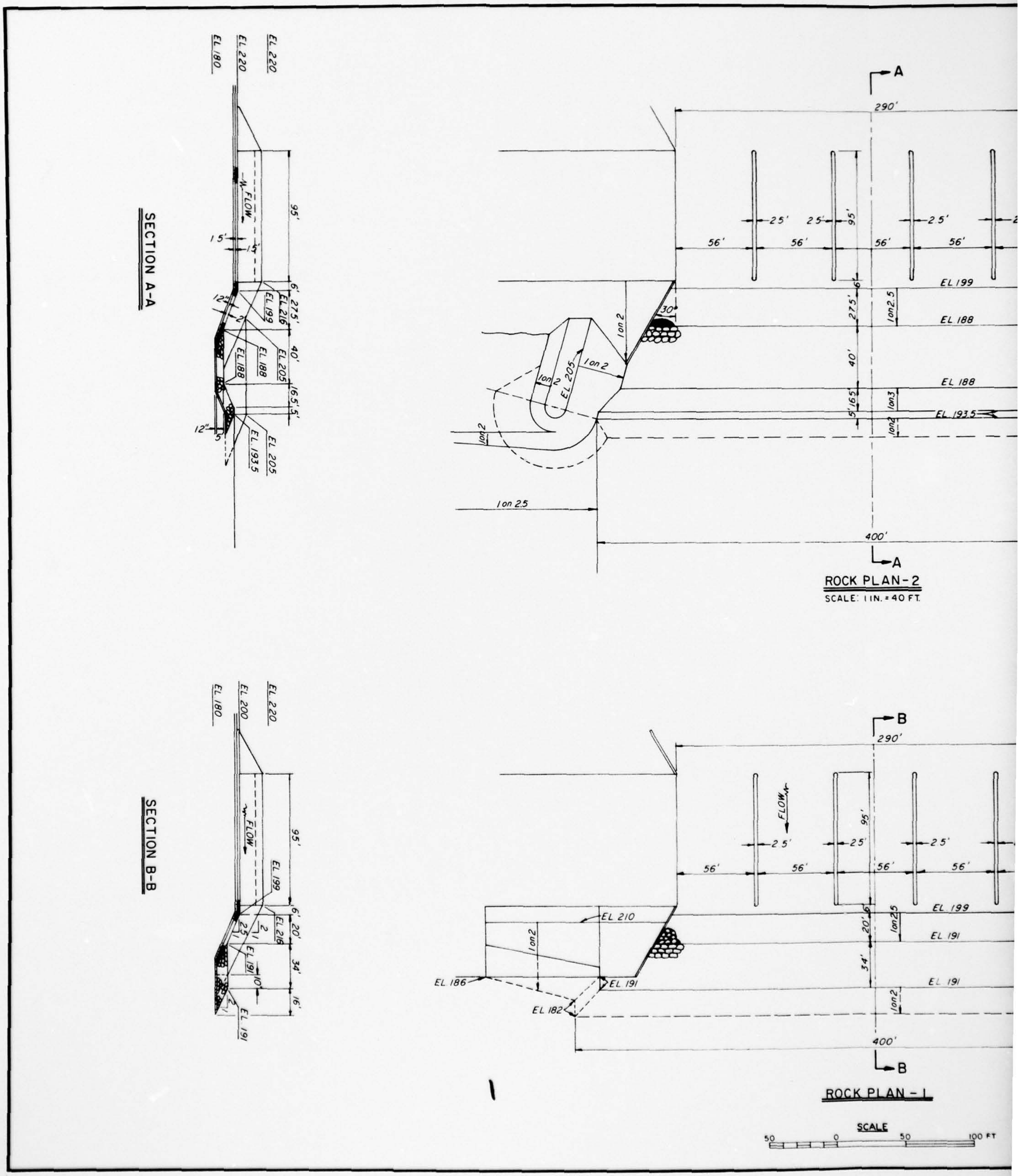


Looking across at downstream side of bridge



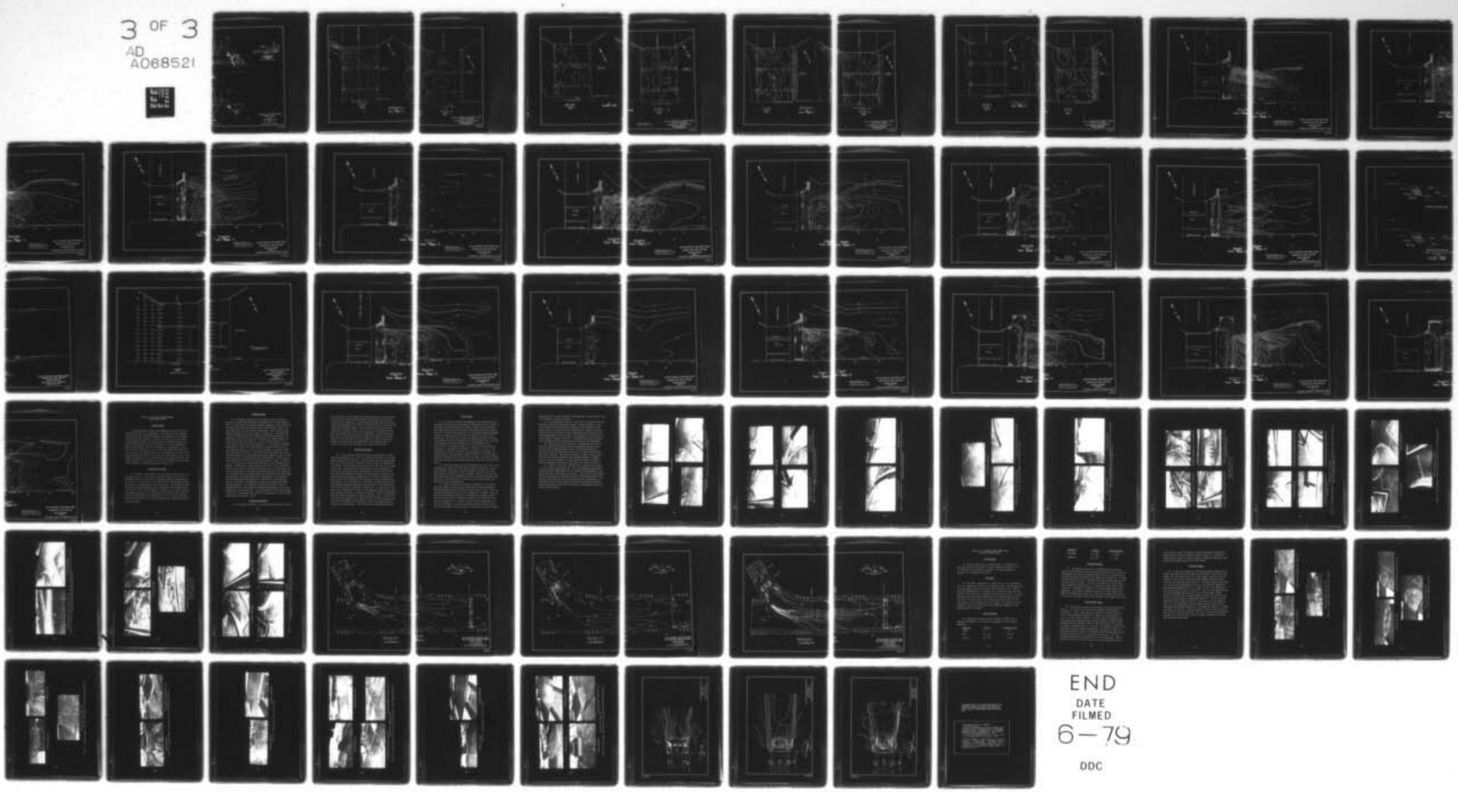
Looking upstream from right side

Photo 62. General model Rosemead drop structure, scale 1:60. Scour downstream of bridge after 38-min run (model); discharge 30,000 cfs through bridge and 10,000 cfs in Rio Hondo, tailwater el 203.5

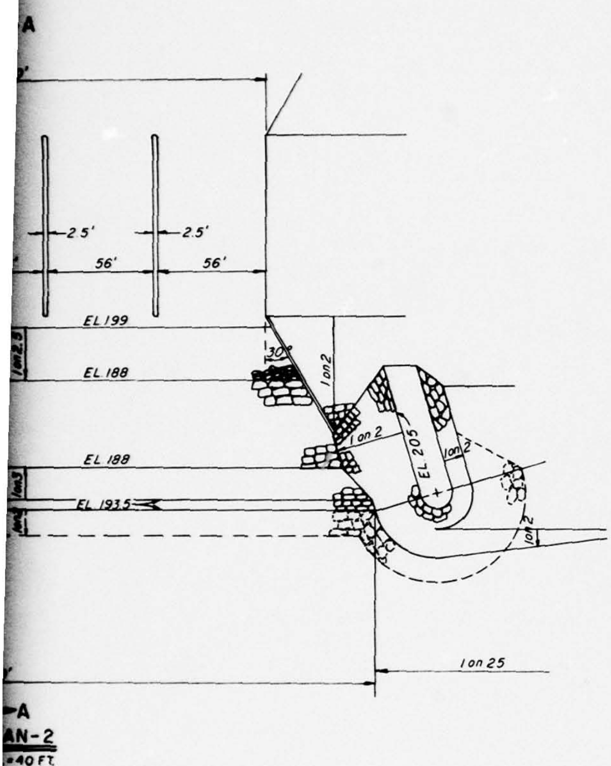

SECTION A-A
SECTION B-B
ROCK PLAN-2
SCALE: 1 IN. = 40 FT.
ROCK PLAN - 1
SCALE
50 0 50 100 FT
FLOW
290'
400'
56'
2.5'
95'
EL 199
EL 188
EL 193.5
EL 205
EL 210
EL 191
EL 186
EL 182
EL 220
EL 200
EL 180
EL 216
1 on 2
1 on 2.5
1 on 3

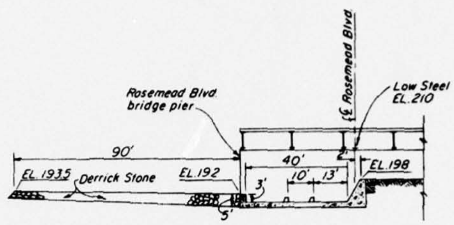
AD-A068 521 ARMY ENGINEER DISTRICT LOS ANGELES CALIF F/G 13/2
WHITTIER NARROWS FLOOD-CONTROL BASIN, LOS ANGELES COUNTY DRAINA--ETC(U)
FEB 79 D A BARELA
UNCLASSIFIED 2-112 NL

3 OF 3
AD
A068521



END
DATE
FILMED
6-79
DDC



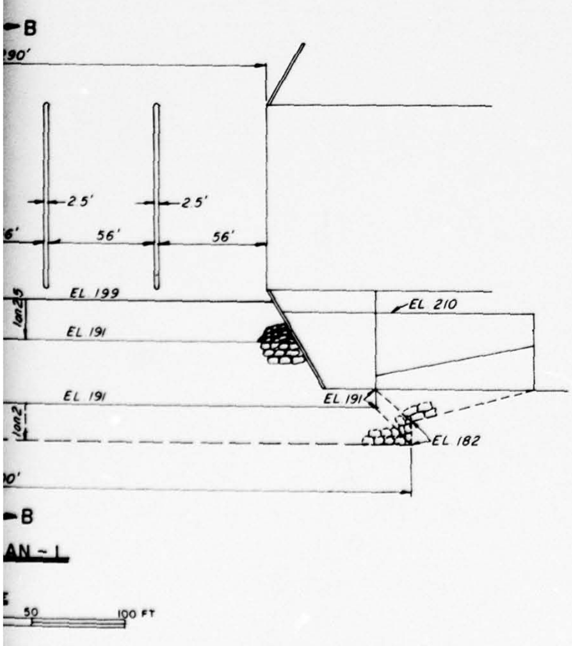


DROP STRUCTURE

Original Design

SCALE

25 0 25 50 75



2

ROSEMEAD BLVD. DROP STRUCTURE
ORIGINAL DESIGN
AND
ALTERNATIVE PLANS
WITH
ROCK PLANS 1 AND 2

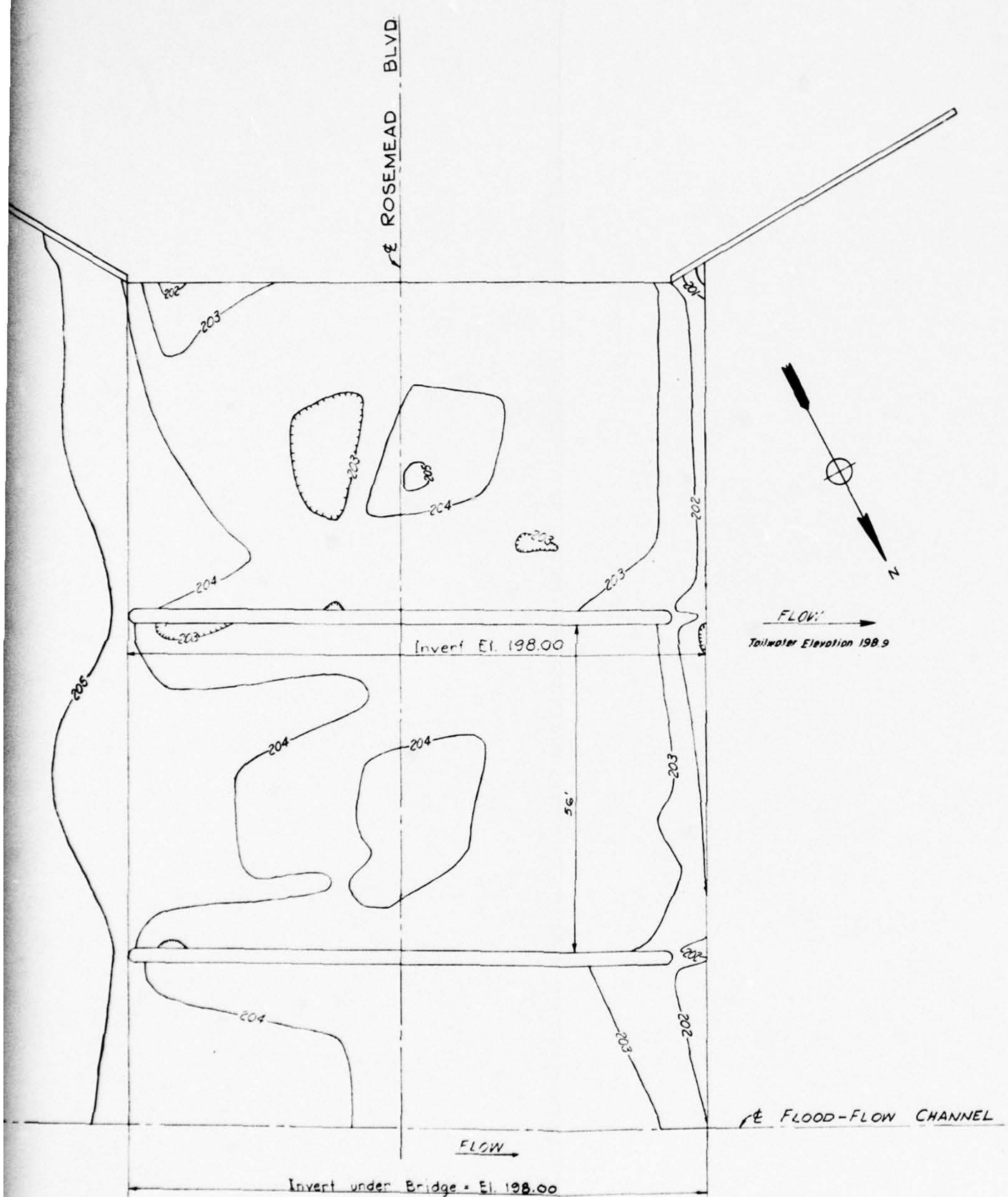
PLATE 40

HALF PLAN

Q = 40,000 cfs

TEST 1

7



FOOT

30FT

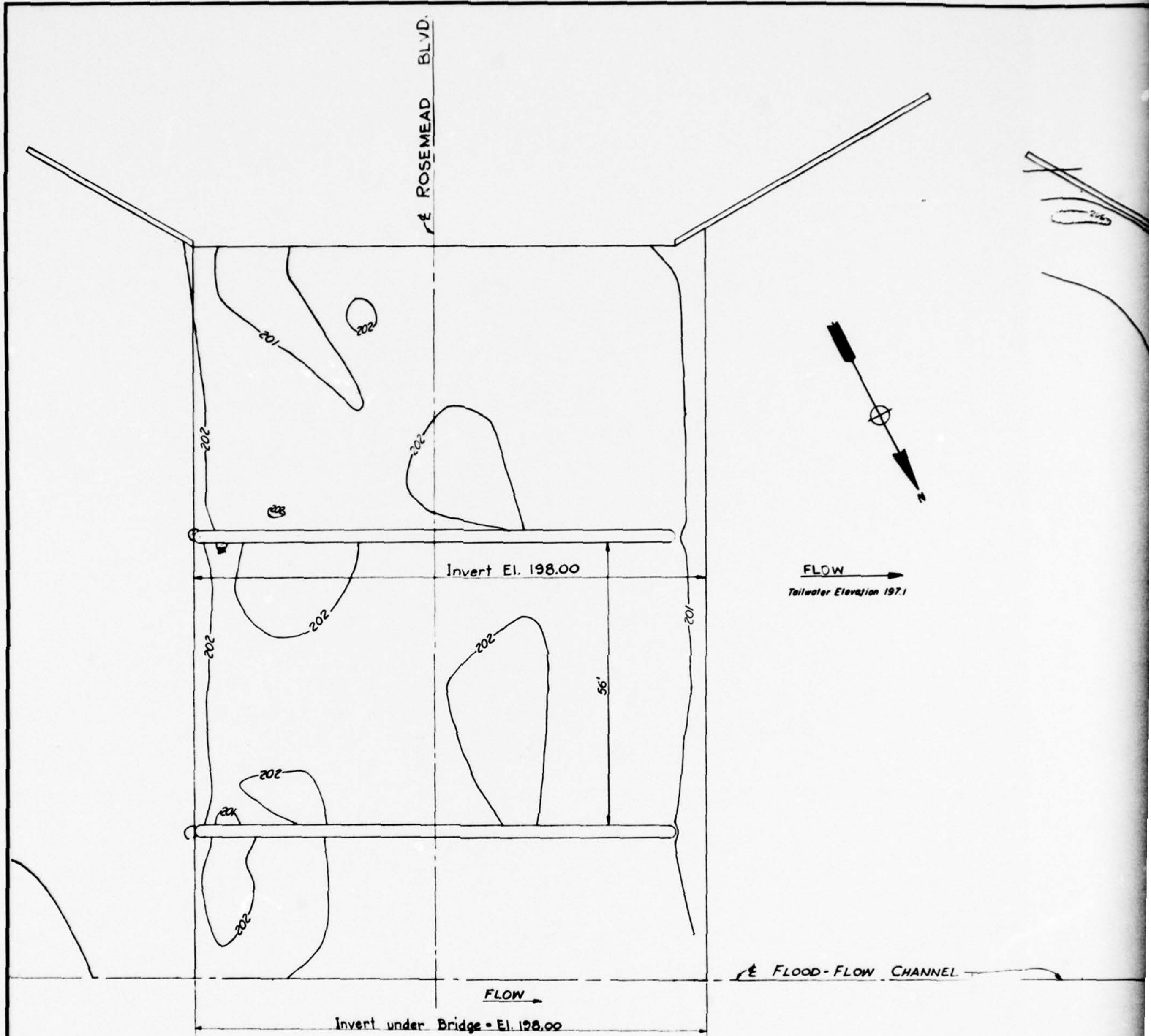
HALF PLAN

Q = 20,000 cfs

TEST 3

WHITTIER NARROWS FLOOD-CONTROL BASIN
FLOOD-FLOW CHANNEL
ROSEMEAD BLVD. DROP STRUCTURE
DEPTH CONTOURS
TESTS 1 AND 3

PLATE 41

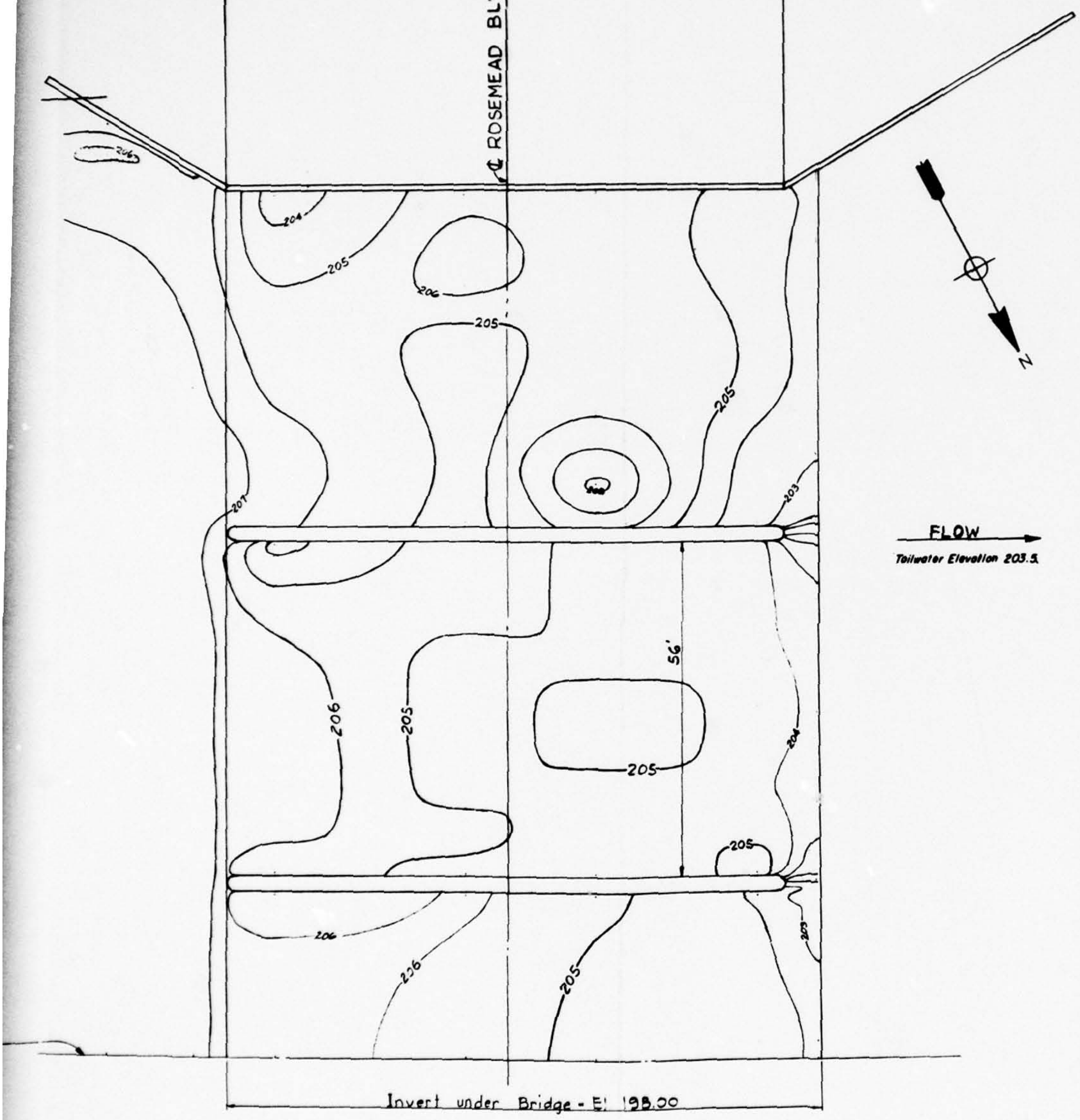


HALF PLAN

Q = 10,000 c.f.s

TEST 4

1



HALF PLAN

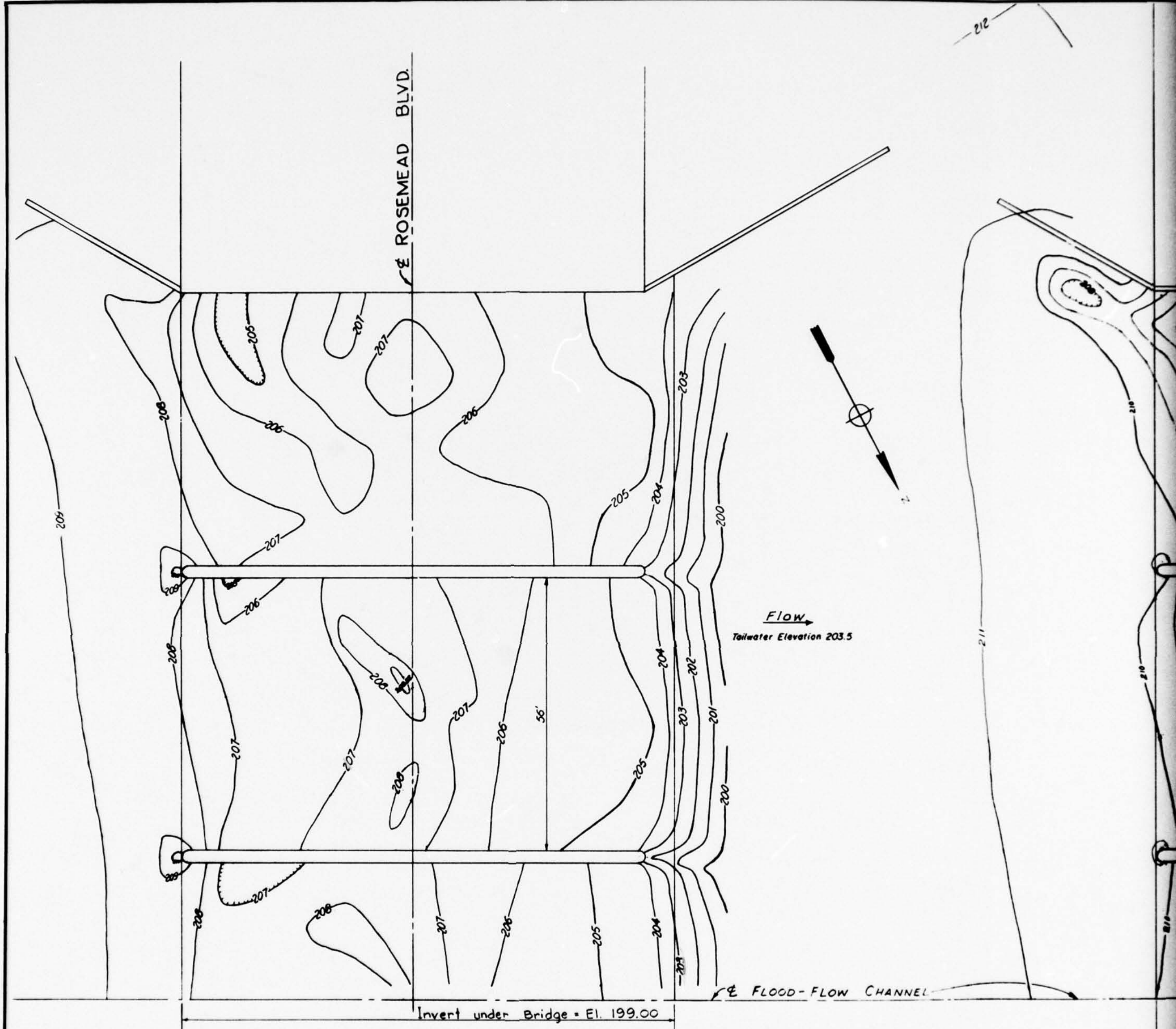
Q = 30,000 c.f.s.

TEST 2

2

DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

WHITTIER NARROWS FLOOD-CONTROL BASIN
FLOOD-FLOW CHANNEL
ROSEMEAD BLVD. DROP STRUCTURE
DEPTH CONTOURS
TESTS 2 AND 4

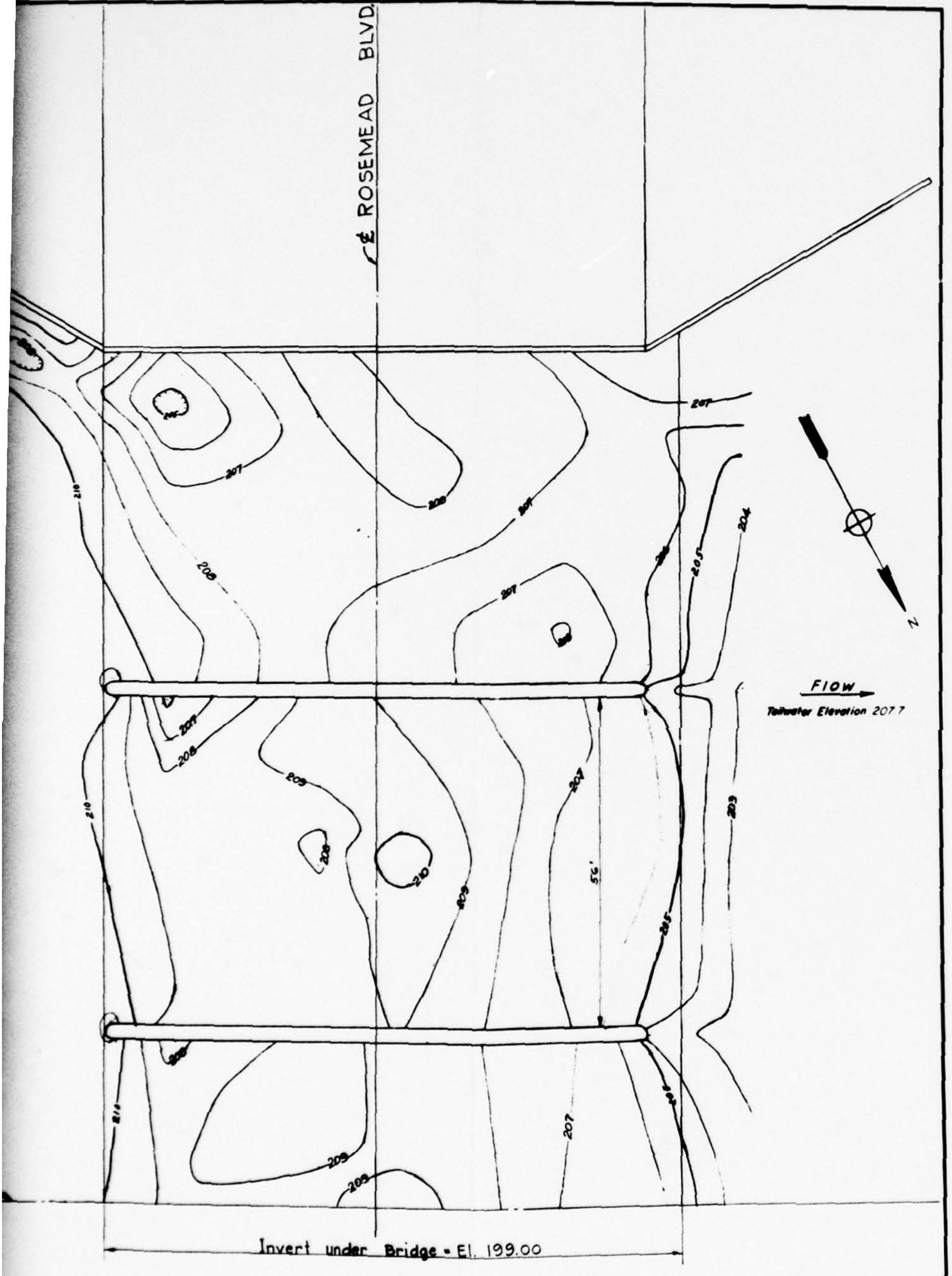


HALF PLAN

Q = 30,000 cfs
TEST 6

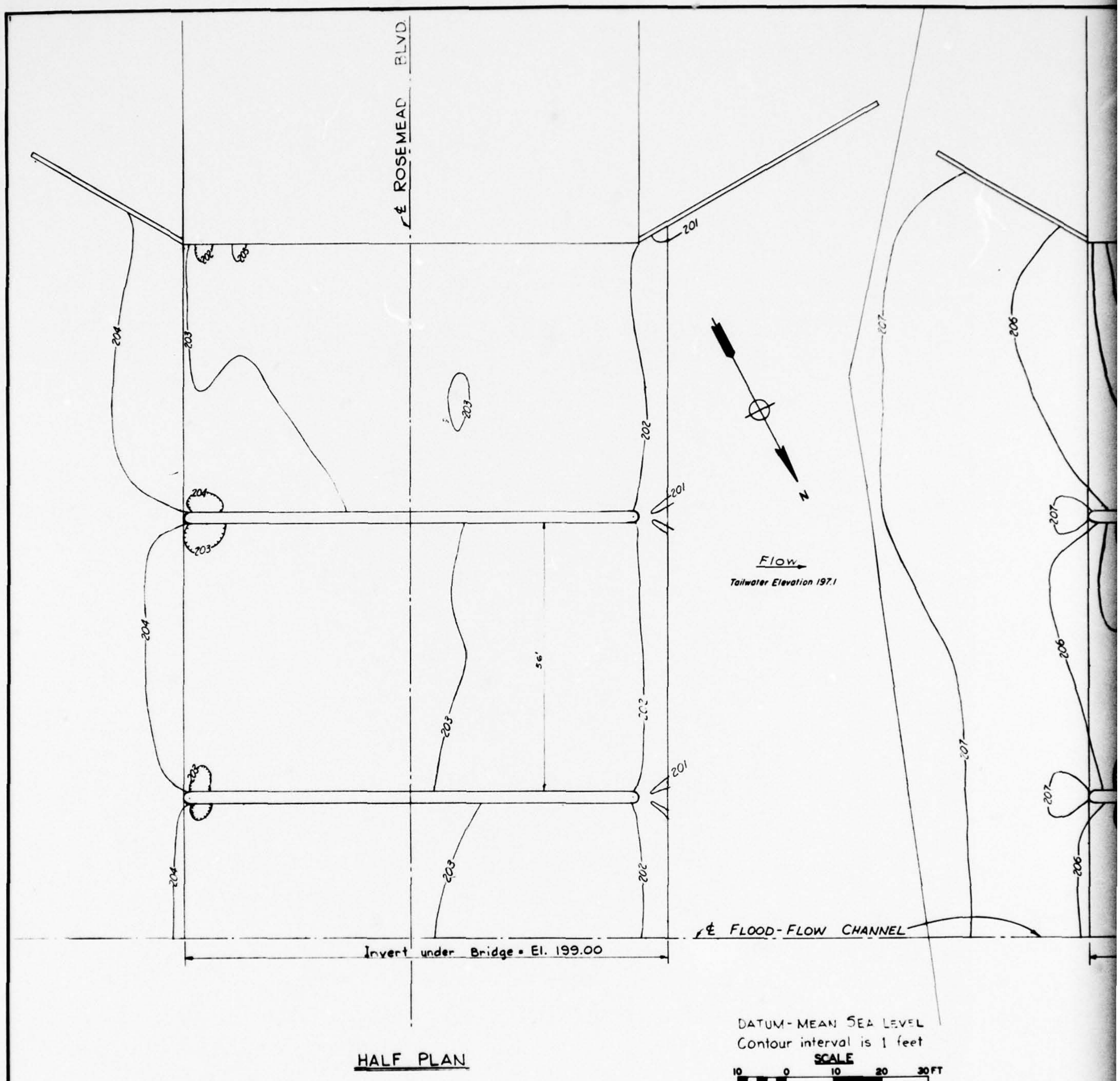
DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

SCALE
10 0 10 20 30 FT



HALF PLAN
Q = 40,000 cfs
TEST 5

WHITTIER NARROWS FLOOD-CONTROL BASIN
FLOOD-FLOW CHANNEL
ROSEMEAD BLVD. DROP STRUCTURE
DEPTH CONTOURS
TESTS 5 AND 6

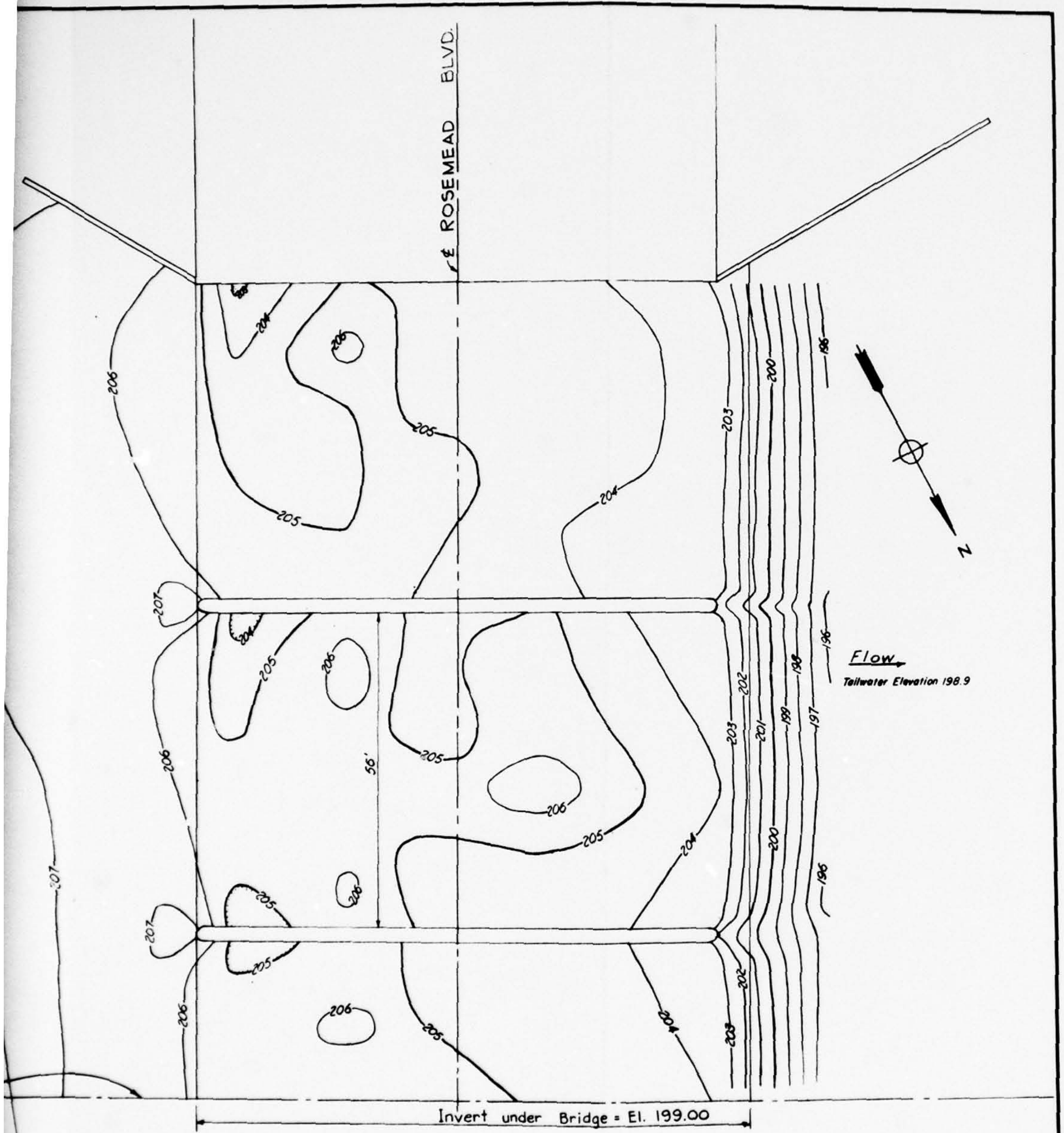


HALF PLAN

Q = 10,000 c.f.s.

TEST 8

1



HALF PLAN

Q = 20,000 c.f.s.

TEST 7

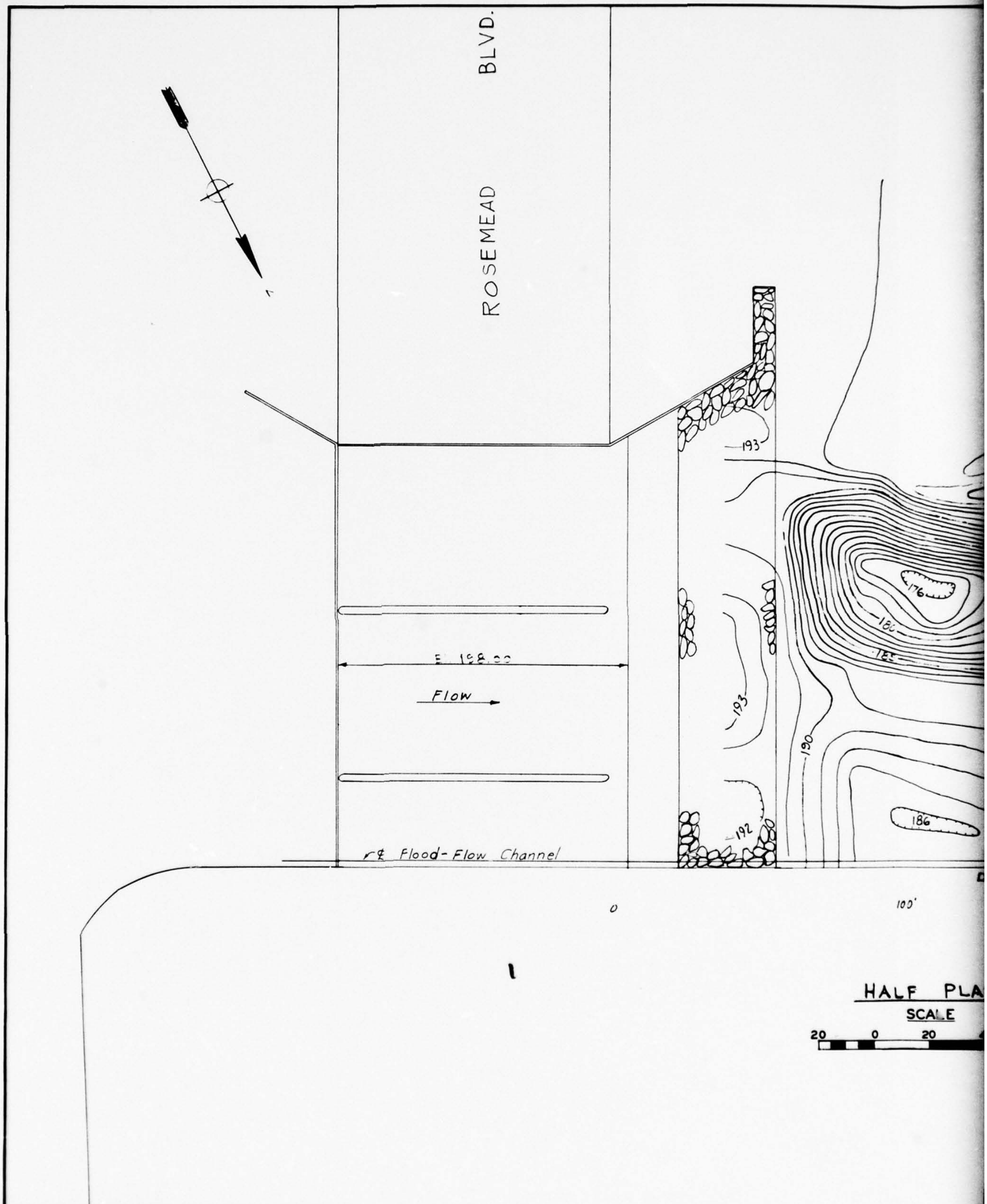
WHITTIER NARROWS FLOOD-CONTROL BASIN

FLOOD-FLOW CHANNEL

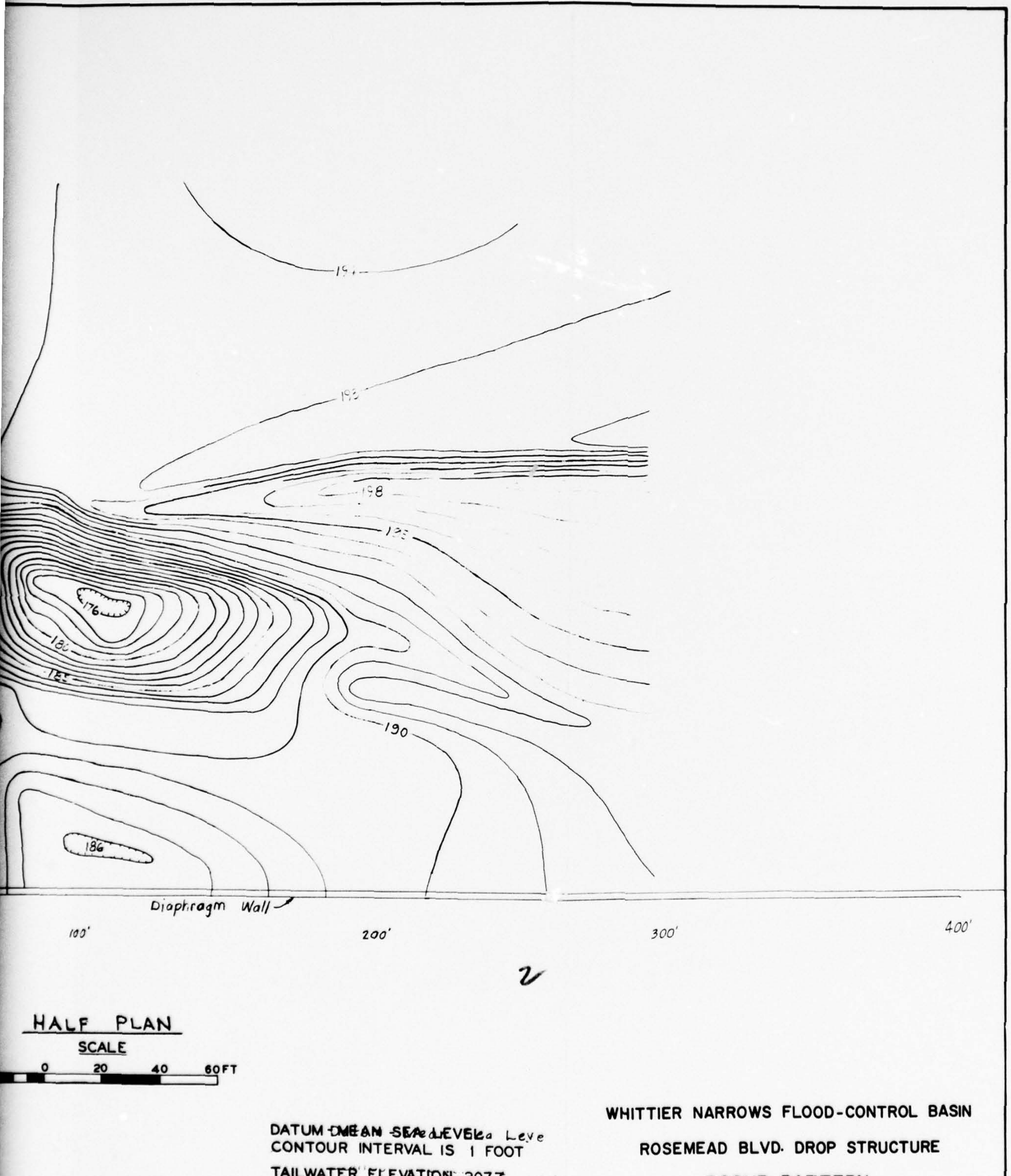
ROSEMEAD BLVD. DROP STRUCTURE

DEPTH CONTOURS

TESTS 7 AND 8



ROSEMEAD BLVD.
Flow
℄ Flood-Flow Channel
193
193
192
190
176
180
185
186
0
100'
HALF PLA
SCALE
20 0 20



HALF PLAN

SCALE

0 20 40 60FT

DATUM ~~MEAN SEA LEVEL~~ Mean Sea Level
CONTOUR INTERVAL IS 1 FOOT

TAILWATER ELEVATION 207.7

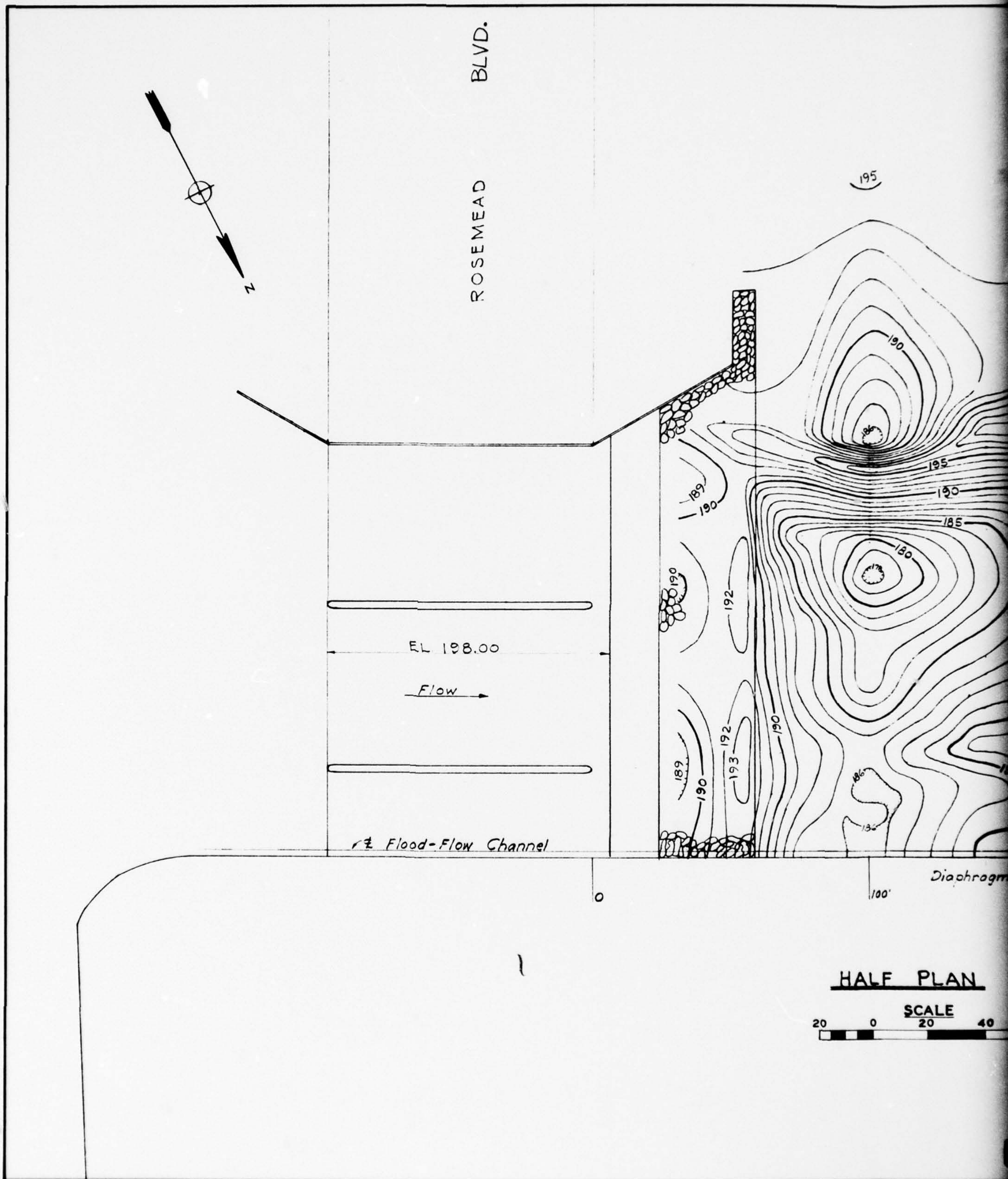
WHITTIER NARROWS FLOOD-CONTROL BASIN

ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN

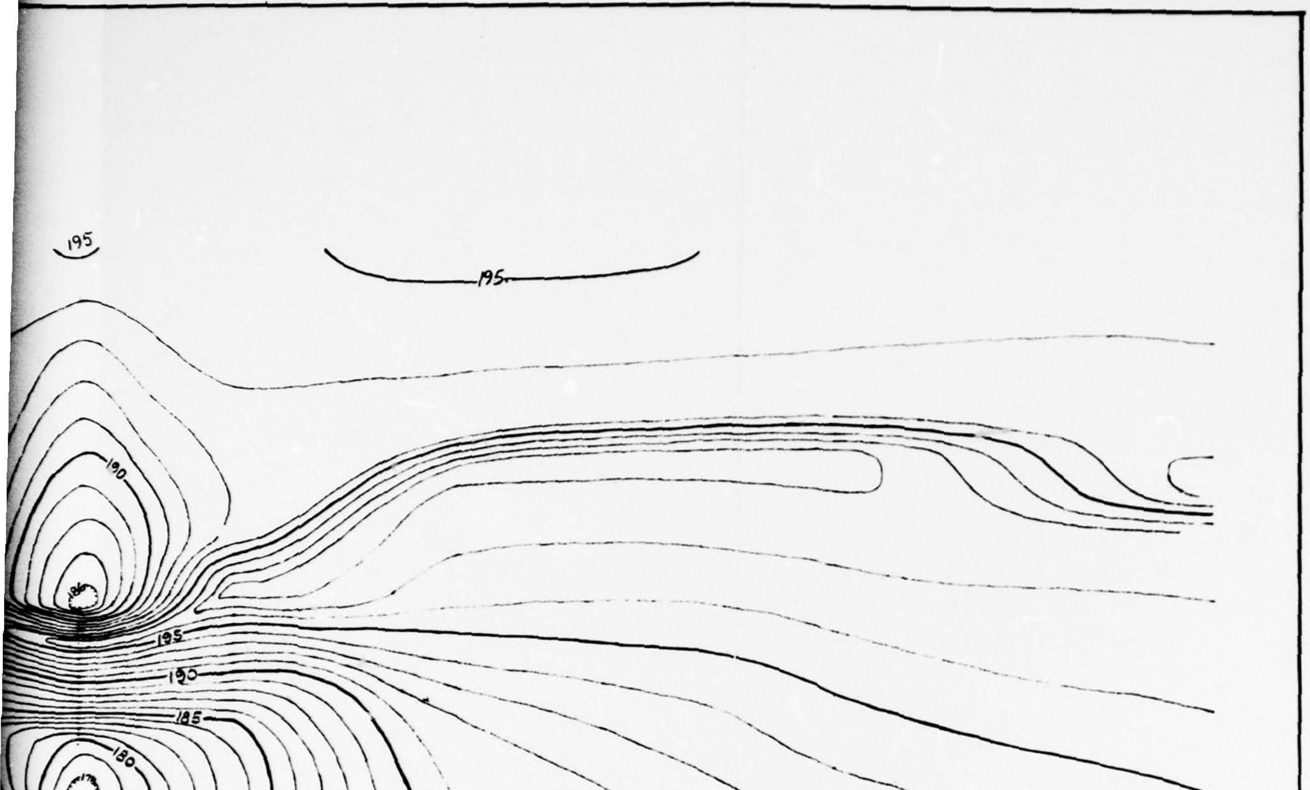
TEST 1

DISCHARGE = 40,000 CFS DURATION = 1 HOUR



HALF PLAN

SCALE
20 0 20 40



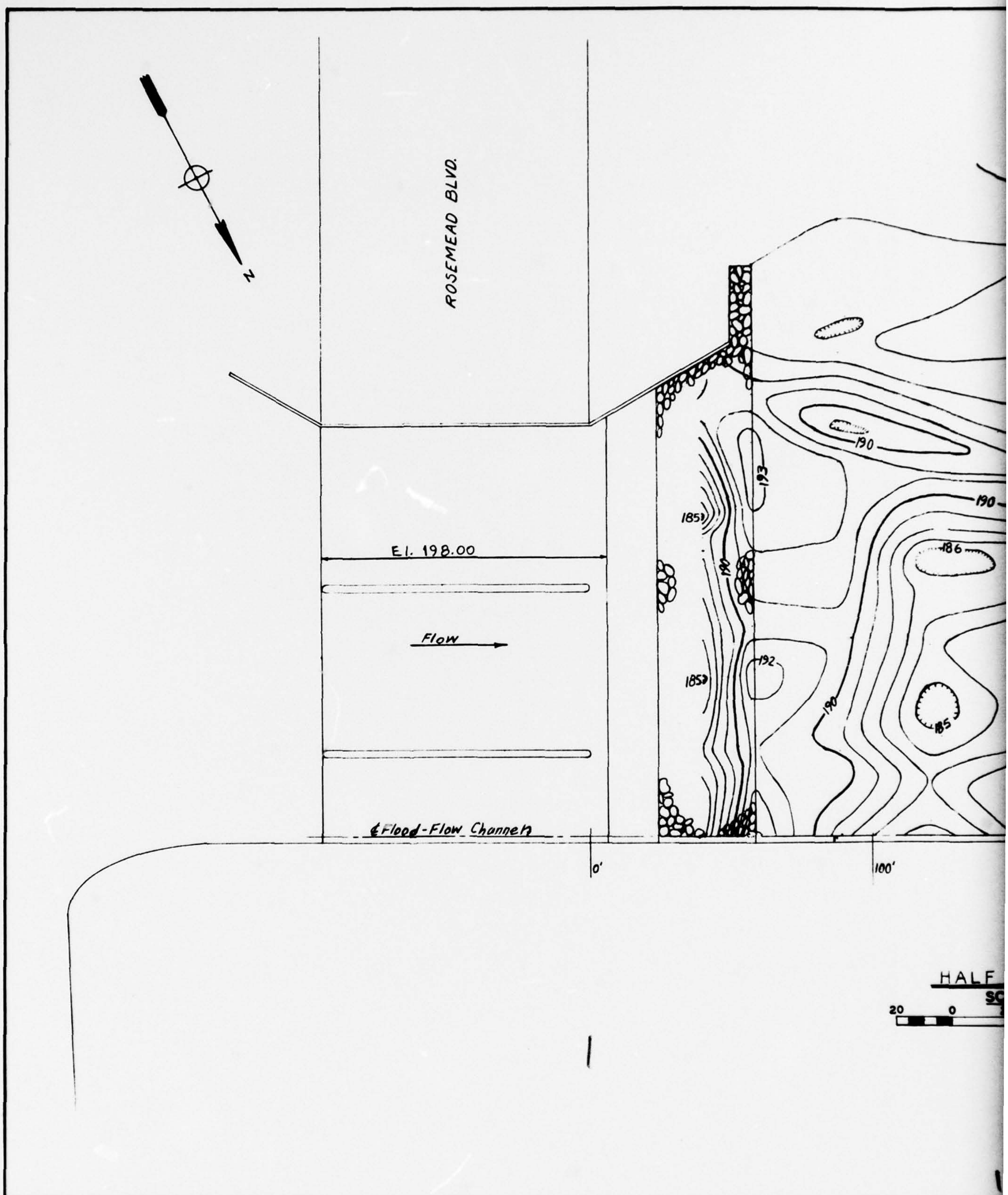
DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

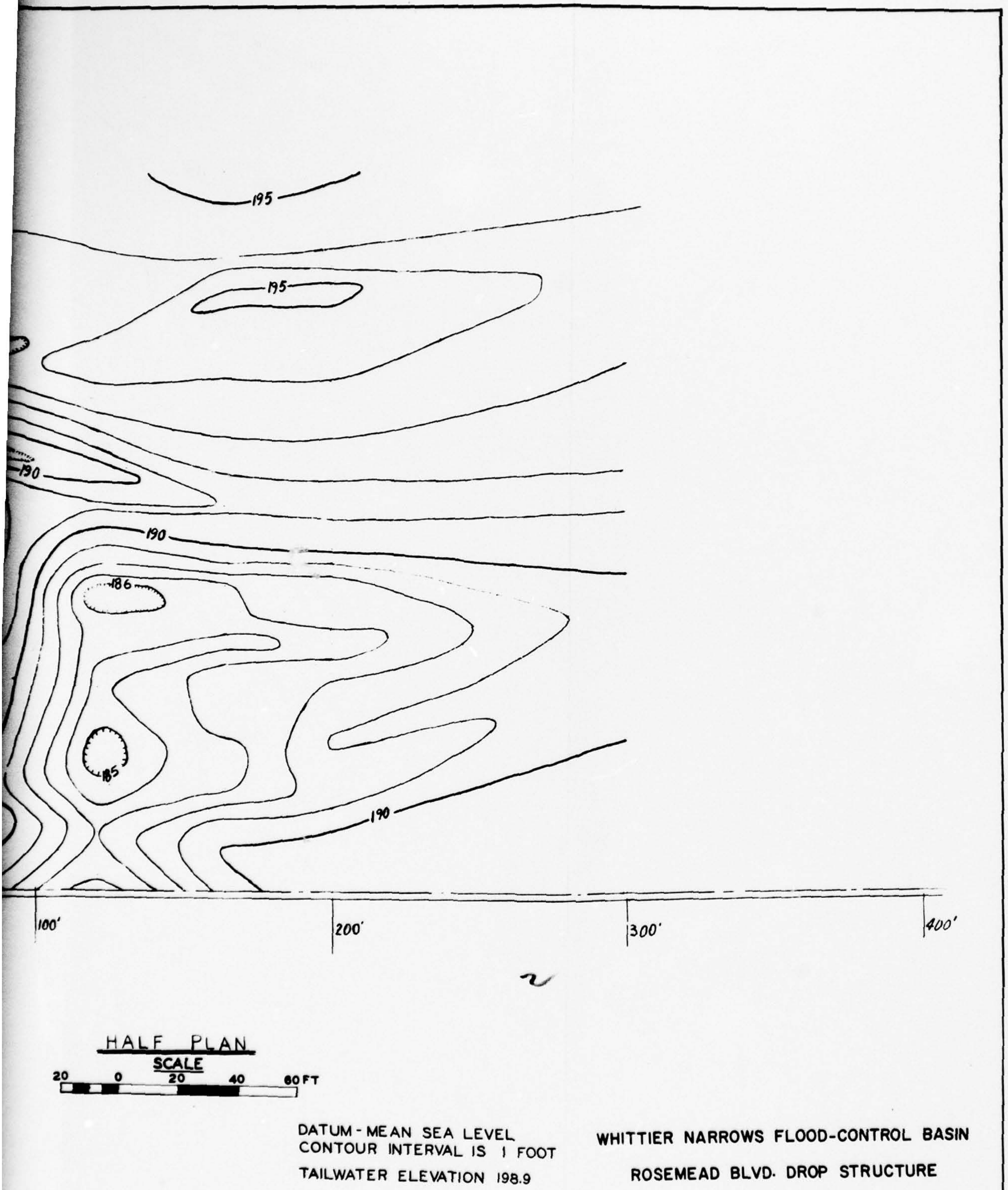
TAILWATER ELEVATION 203.5

WHITTIER NARROWS FLOOD-CONTROL BASIN

ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN
TEST 2
DISCHARGE = 30,000 CFS, DURATION = 1 HOUR


N
ROSEMEAD BLVD.
El. 198.00
Flow
℄ Flood-Flow Channel
0'
100'
190
193
185
190
192
185
190
190
186
185
HALF
20
0



HALF PLAN

SCALE

20 0 20 40 60 FT

DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

TAILWATER ELEVATION 198.9

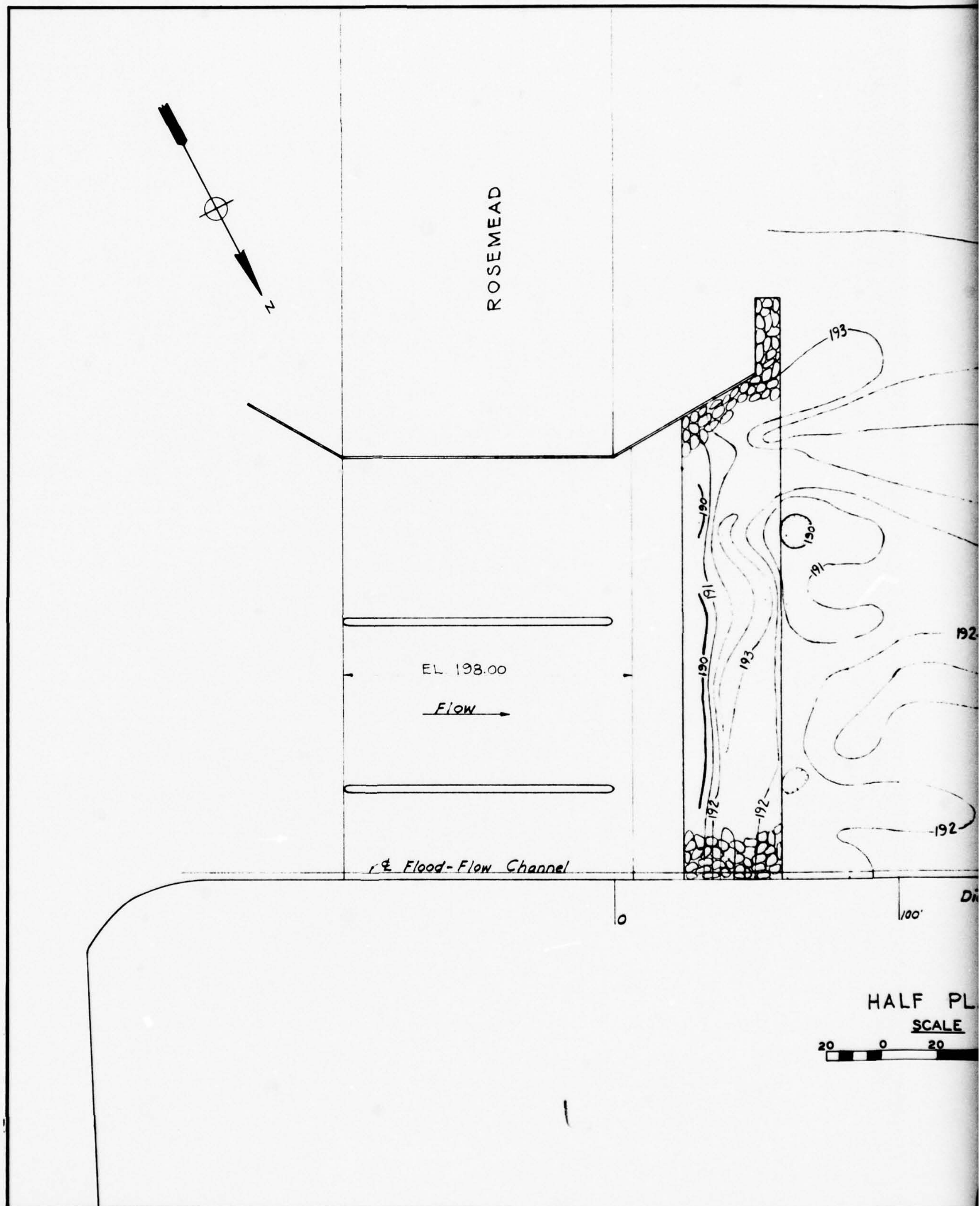
WHITTIER NARROWS FLOOD-CONTROL BASIN

ROSEMEAD BLVD. DROP STRUCTURE

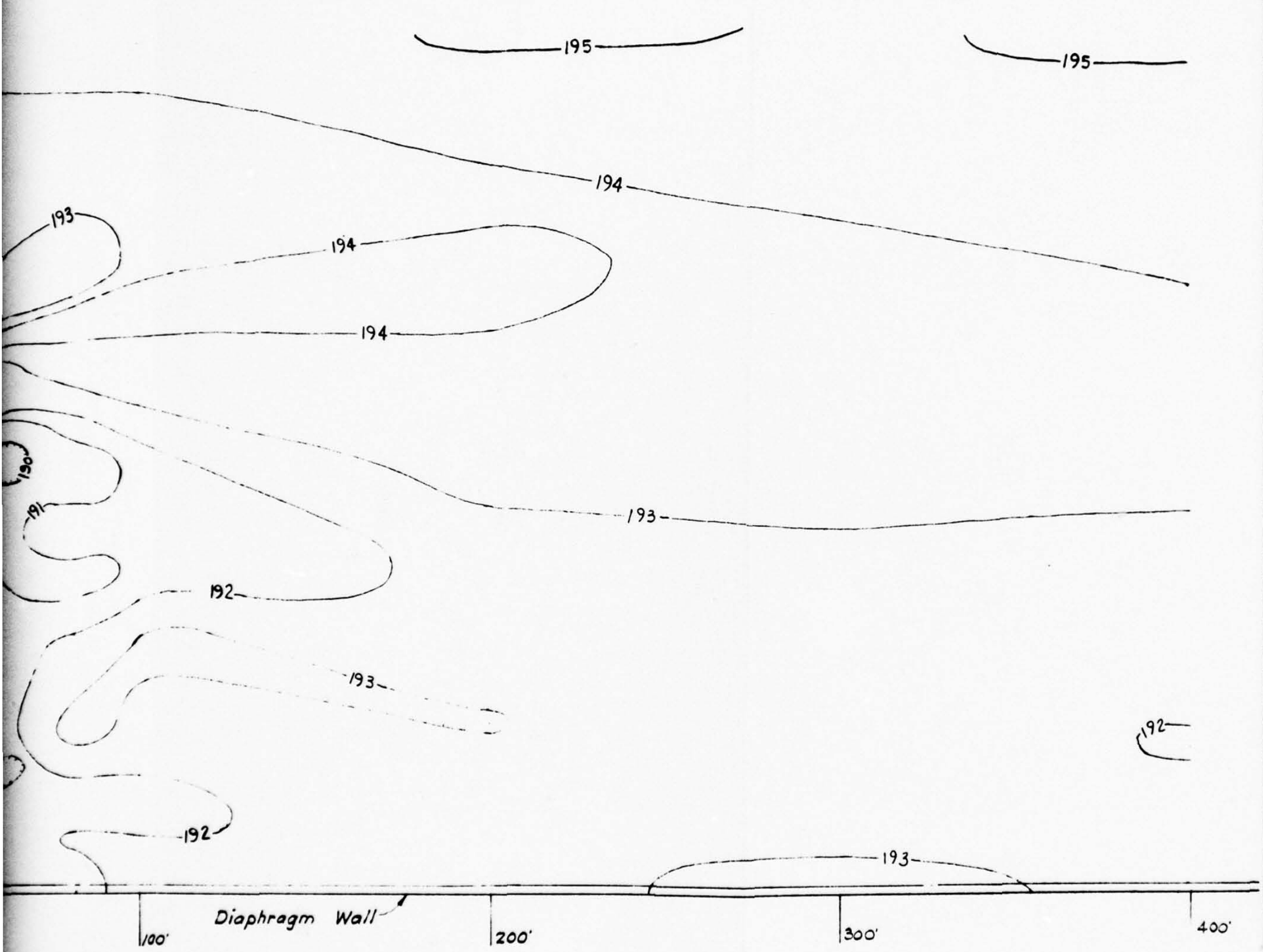
SCOUR PATTERN

TEST 3

DISCHARGE = 20,000 CFS, DURATION = 1 HOUR



ROSEMEAD
N
193
190
191
192
EL 198.00
Flow
℄ Flood-Flow Channel
0
100'
HALF PL
SCALE
20 0 20



HALF PLAN

SCALE

20 0 20 40 60 FT

DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

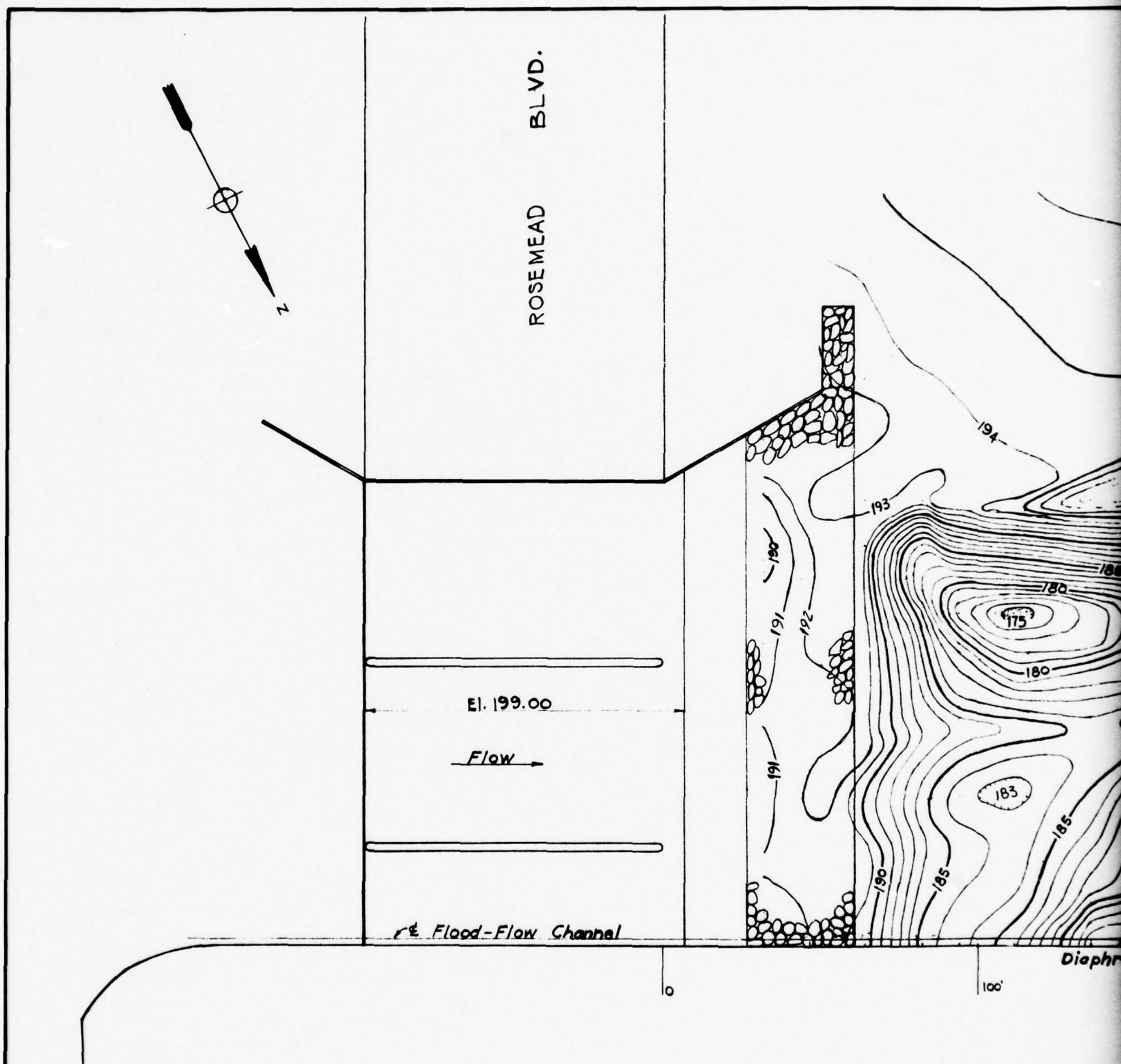
TAILWATER ELEVATION 197.1

WHITTIER NARROWS FLOOD-CONTROL BASIN

ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN
TEST 4

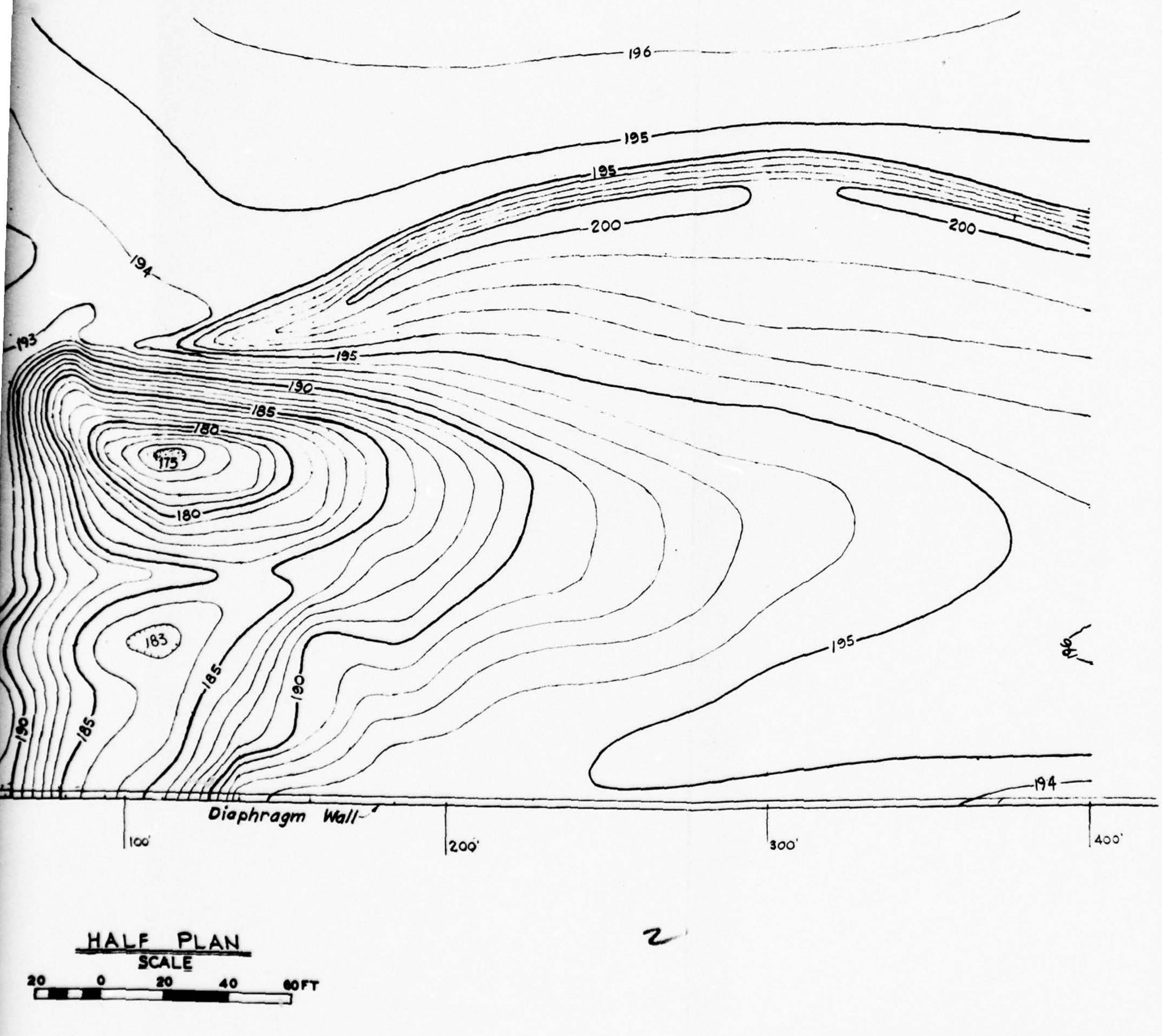
DISCHARGE = 10,000 CFS, DURATION = 1 HOUR



HALF PLAN

SCALE

20 0 20 40



DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT
TAILWATER ELEVATION 2077

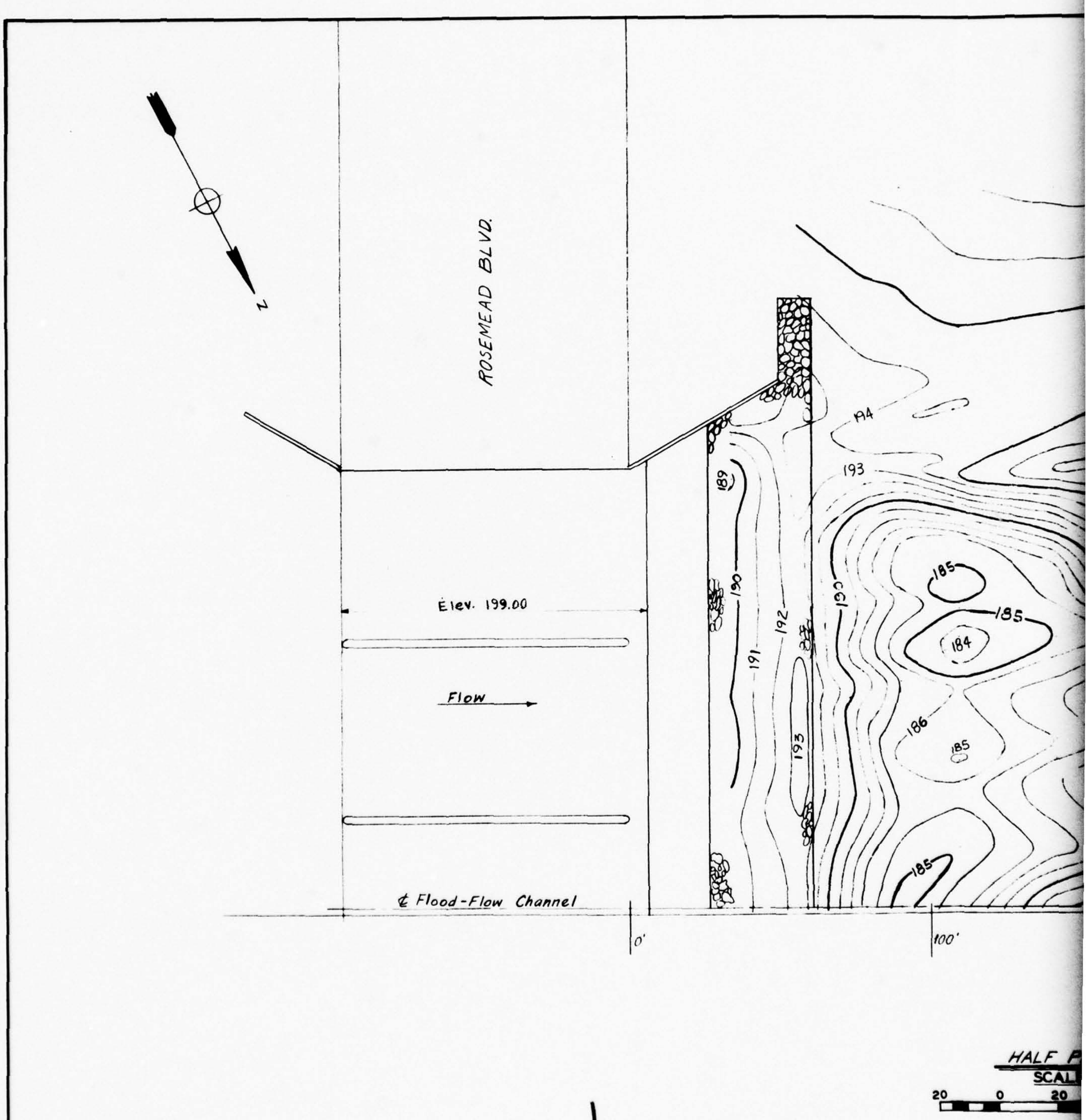
WHITTIER NARROWS FLOOD-CONTROL BASIN

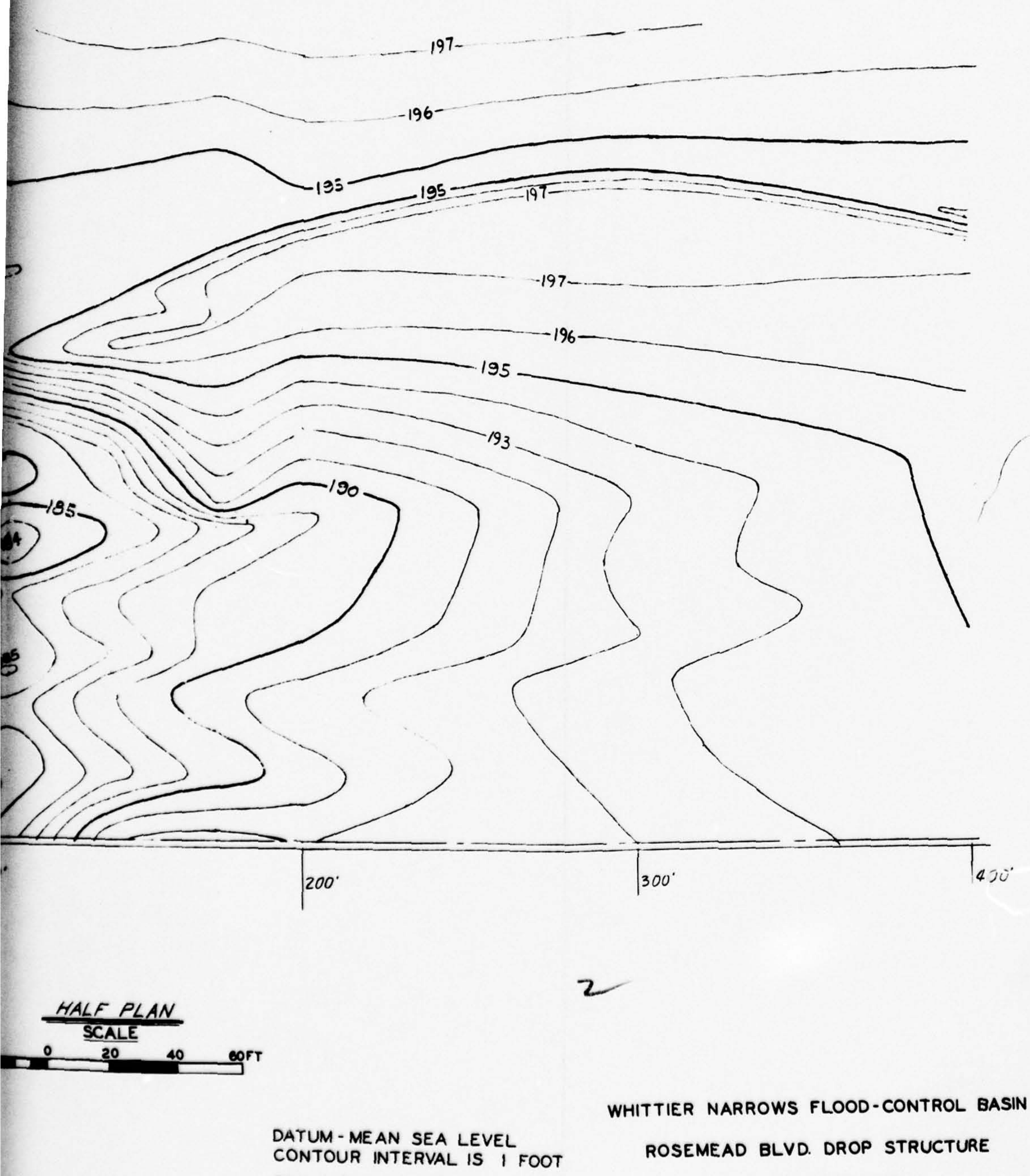
ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN

TEST 5

DISCHARGE = 40,000 CFS, DURATION = 1 HOUR


N
ROSEMEAD BLVD.
Elev. 199.00
Flow
℄ Flood-Flow Channel
0'
100'
194
193
189
190
191
192
193
185
185
184
186
185
185
HALF P
SCAL
20
0
20
1



HALF PLAN

SCALE

0 20 40 60FT

DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

TAILWATER ELEVATION 203.5

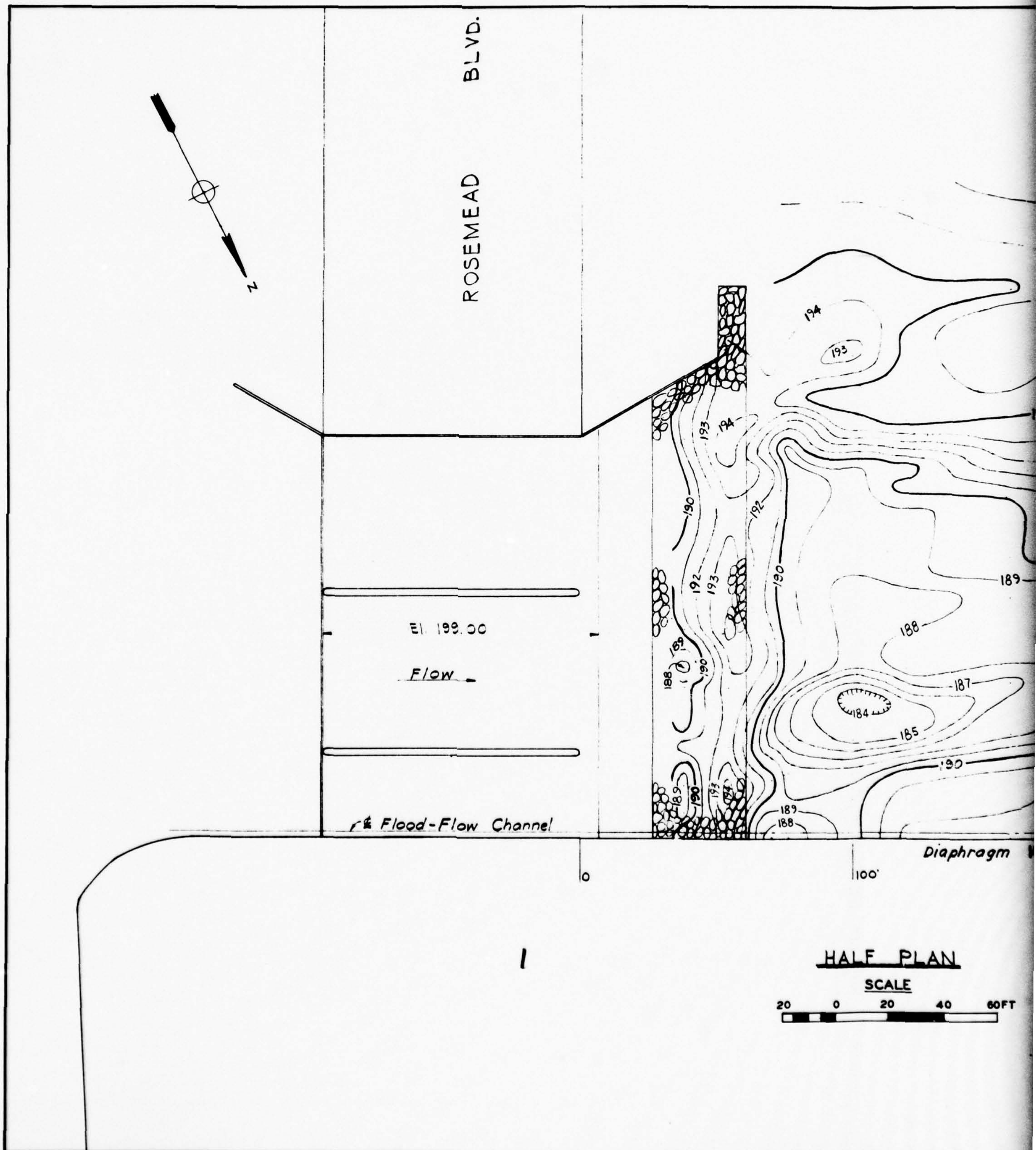
WHITTIER NARROWS FLOOD-CONTROL BASIN

ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN

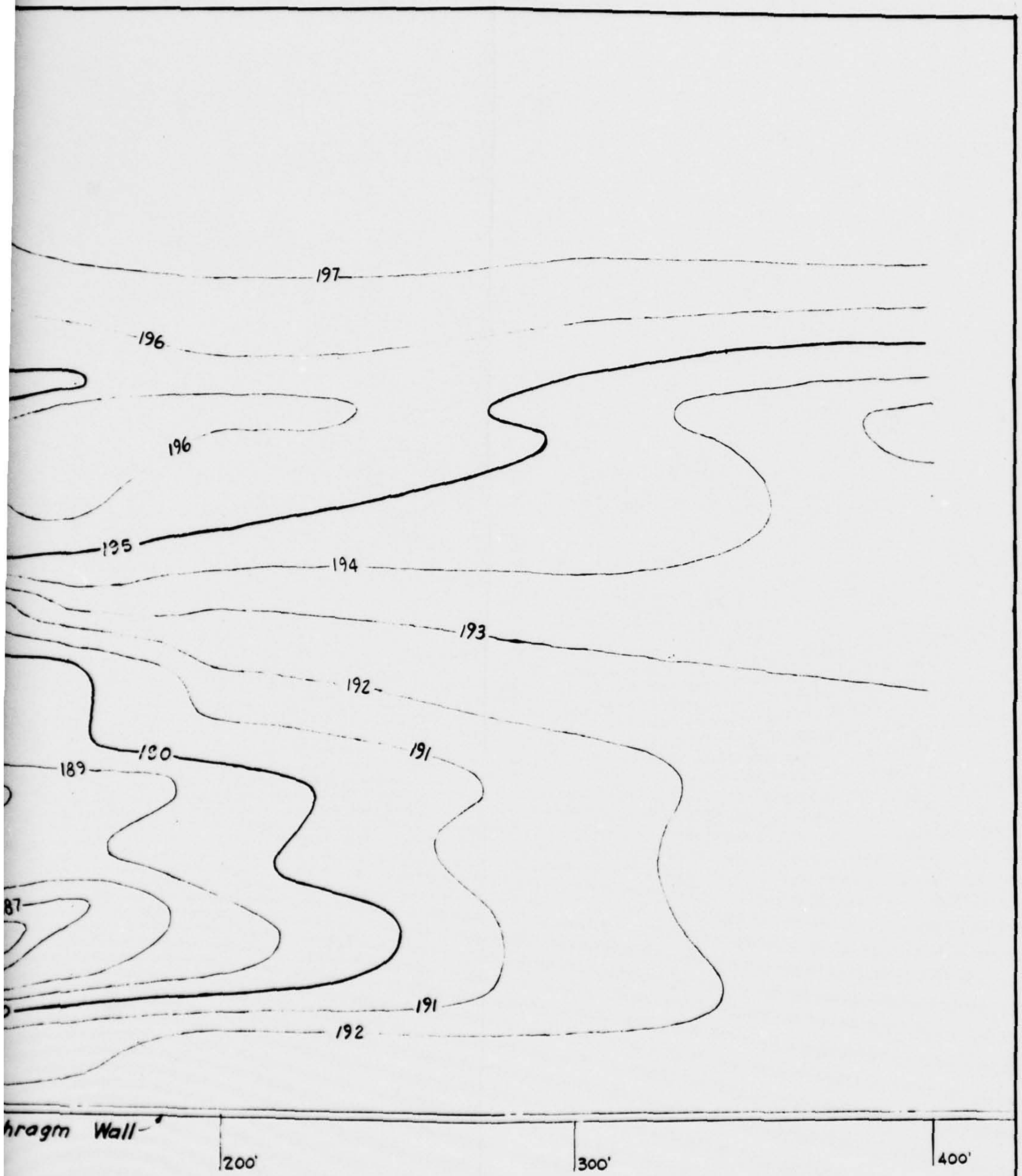
TEST 6

DISCHARGE = 30,000 CFS, DURATION = 1 HOUR



ROSEMEAD BLVD.
N
El. 199.00
Flow
℄ Flood-Flow Channel
Diaphragm
0
100'
194
193
192
190
189
188
187
185
184
HALF PLAN
SCALE
20 0 20 40 60FT

1



2

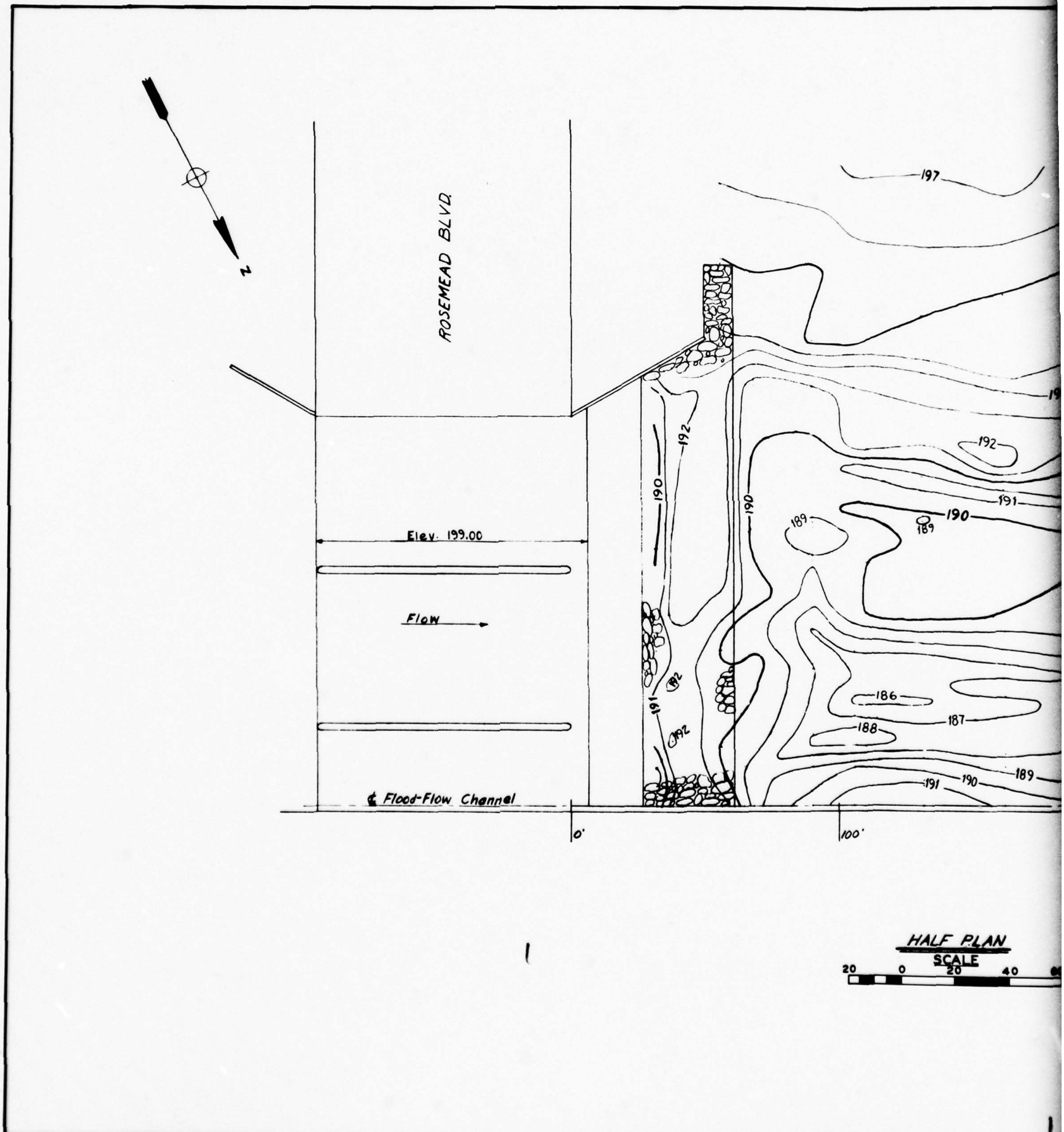
DA... SEA LEVEL
CONTO... RVAL IS 1 FOOT
TAILWATER ELEVATION 198.9

WHITTIER NARROWS FLOOD-CONTROL BASIN

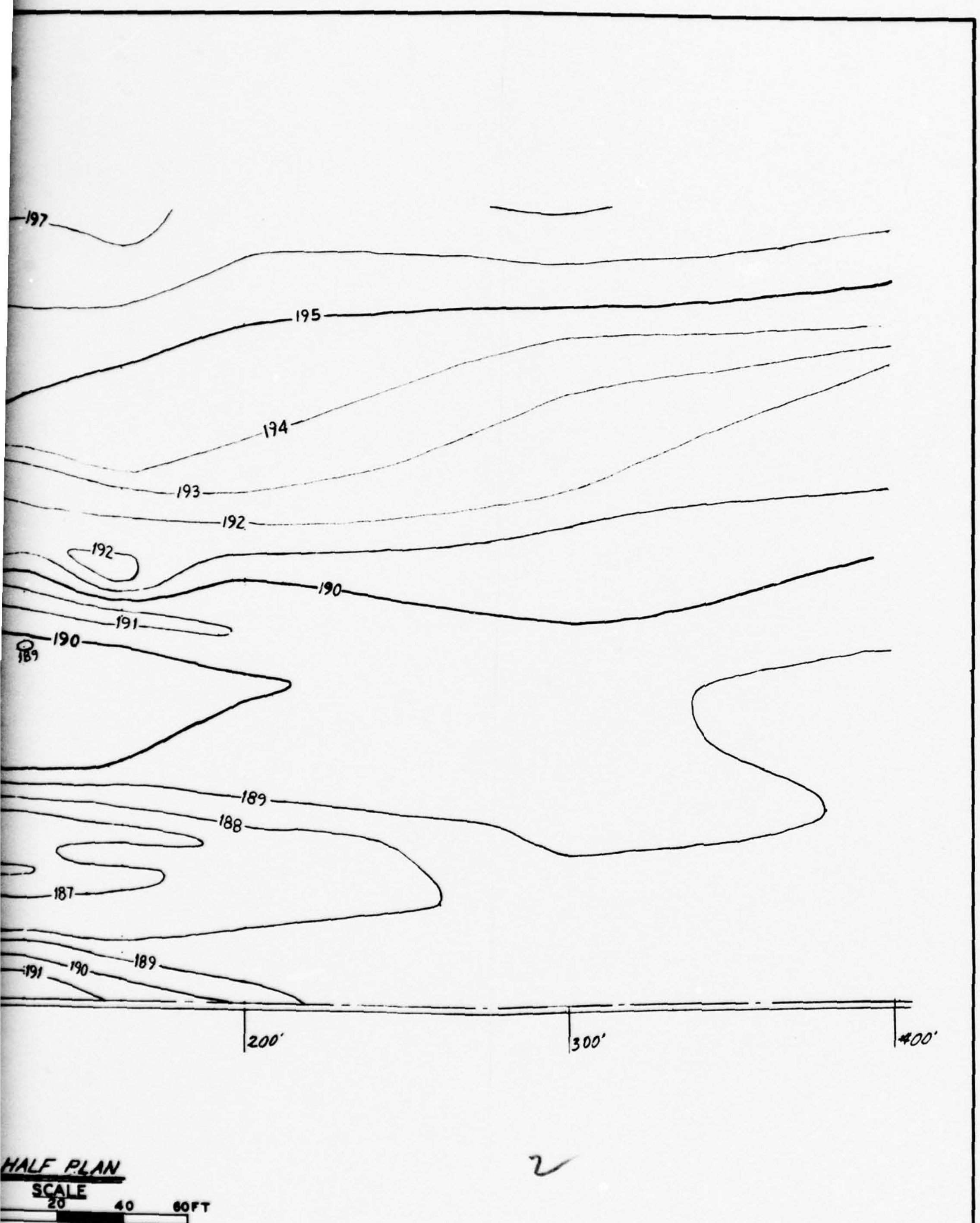
ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN
TEST 7

DISCHARGE = 20,000 CFS, DURATION = 1 HOUR

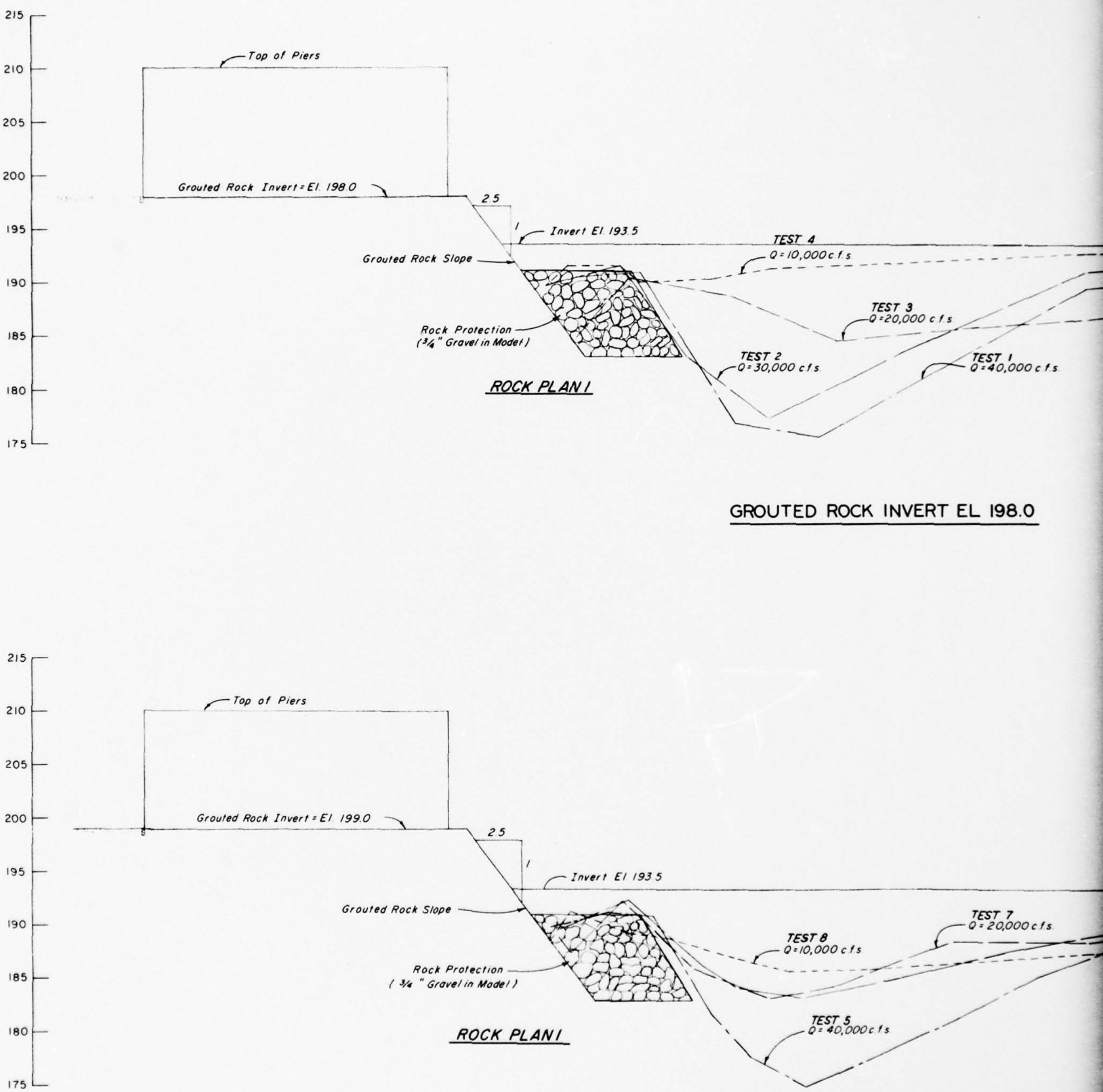


N
ROSEMEAD BLVD.
Elev. 199.00
Flow
℄ Flood-Flow Channel
0'
100'
197
192
190
191
189
186
187
188
HALF PLAN
SCALE
20
0
20
40



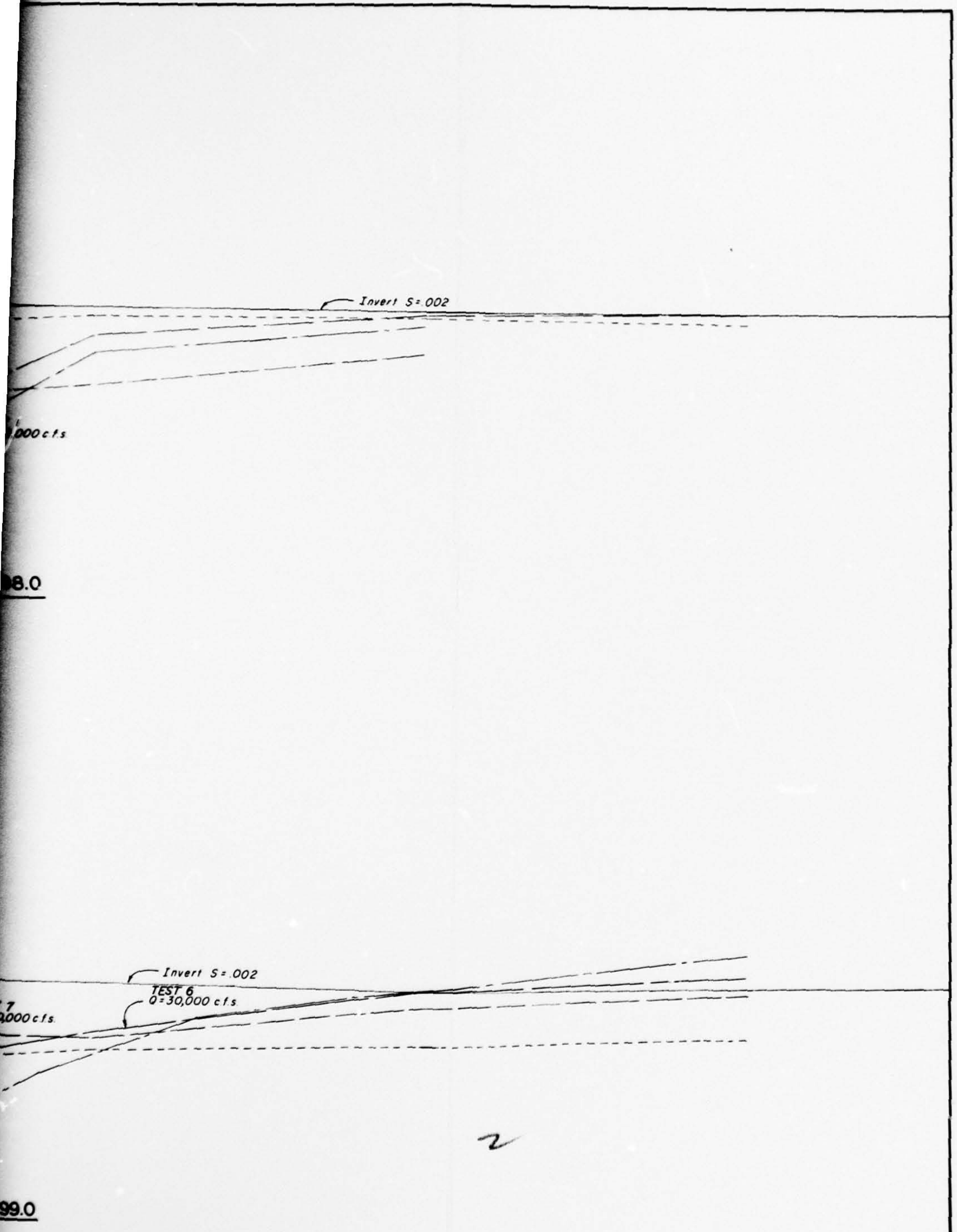
DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT
TAILWATER ELEVATION 197.1

WHITTIER NARROWS FLOOD-CONTROL BASIN
ROSEMEAD BLVD. DROP STRUCTURE
SCOUR PATTERN
TEST 8
DISCHARGE = 10,000 CFS, DURATION = 1 HOUR

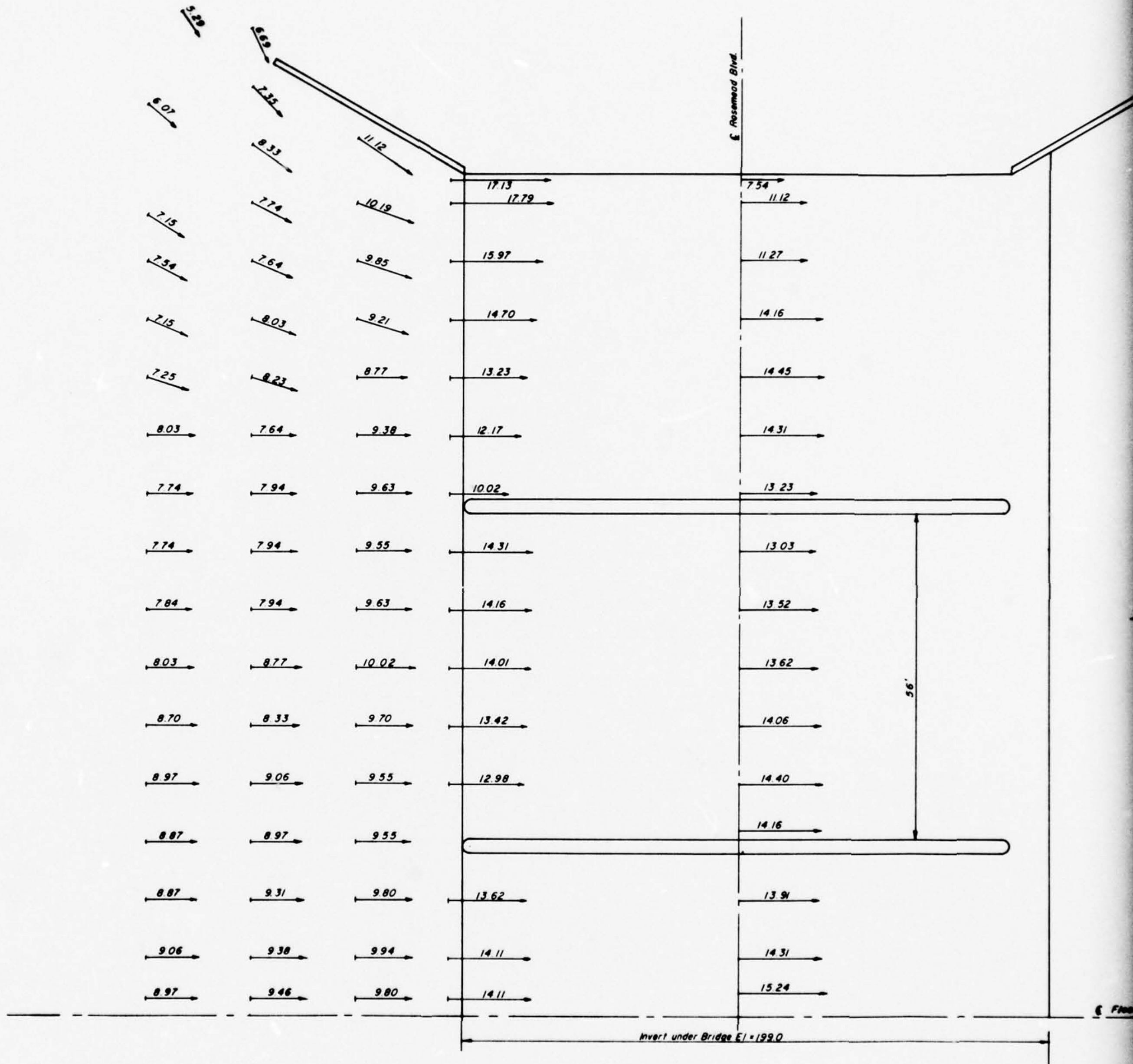


PROFILES OF LOW POINTS OF SCOUR PATTERN
AFTER 1 HR. FLOW

SCALES
HORIZ 20 0 20 40FT
VERT 6 0 6 12FT



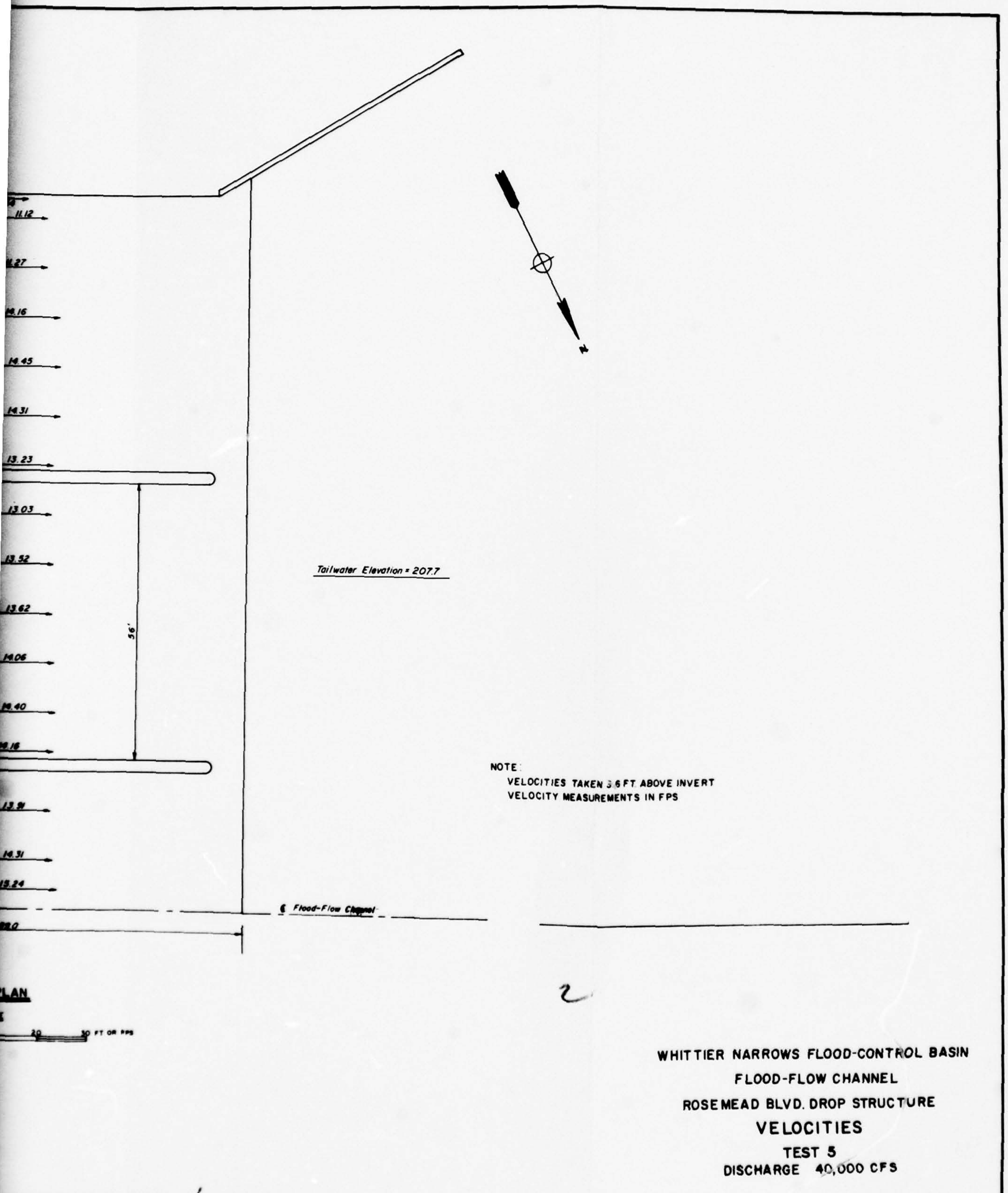
WHITTIER NARROWS FLOOD-CONTROL BASIN
ROSEMEAD BLVD. DROP STRUCTURE
SCOUR PROFILES
BRIDGE INVERT AT EL. 198 AND 199
MODEL RATIO = 1:24



HALF PLAN

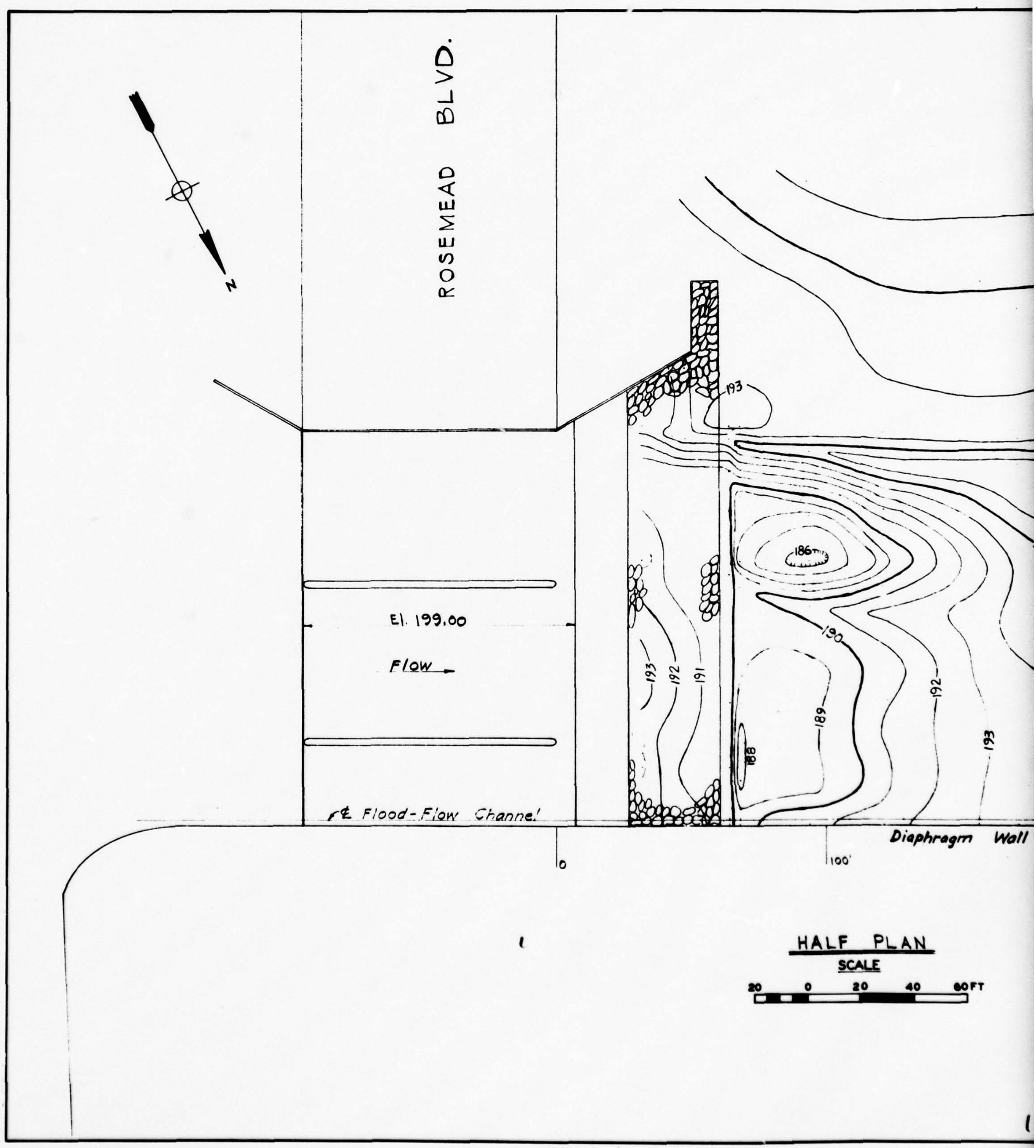
SCALE

10 0 10 20 30 FT OR FPS

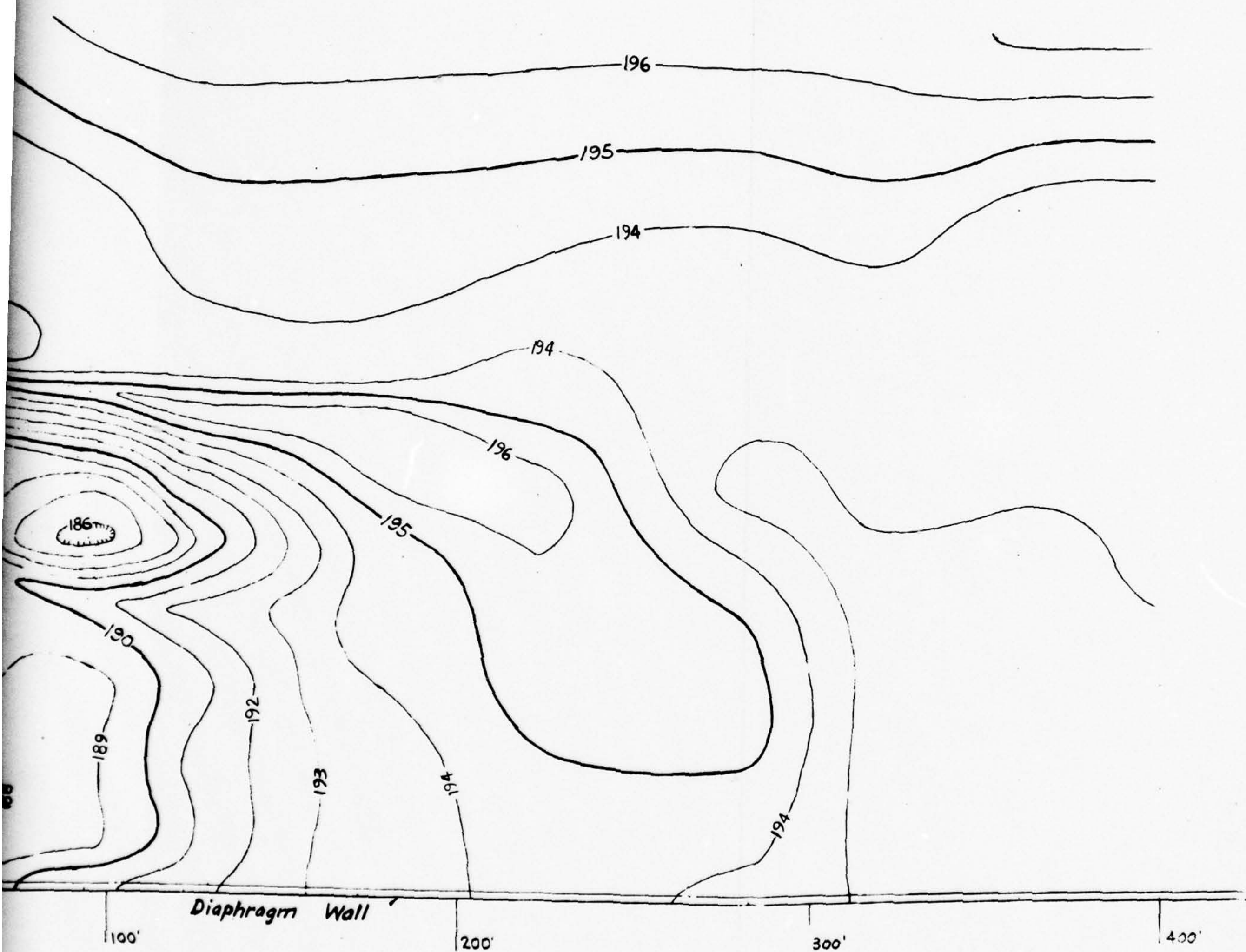


NOTE:
VELOCITIES TAKEN 3.6 FT ABOVE INVERT
VELOCITY MEASUREMENTS IN FPS

WHITTIER NARROWS FLOOD-CONTROL BASIN
FLOOD-FLOW CHANNEL
ROSEMEAD BLVD. DROP STRUCTURE
VELOCITIES
TEST 5
DISCHARGE 40,000 CFS



N
ROSEMEAD BLVD.
El. 199.00
Flow
℄ Flood-Flow Channel
193
192
191
186
190
189
188
Diaphragm Wall
0
100'
HALF PLAN
SCALE
20 0 20 40 60FT



HALF PLAN

SCALE

0 20 40 60 FT

DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

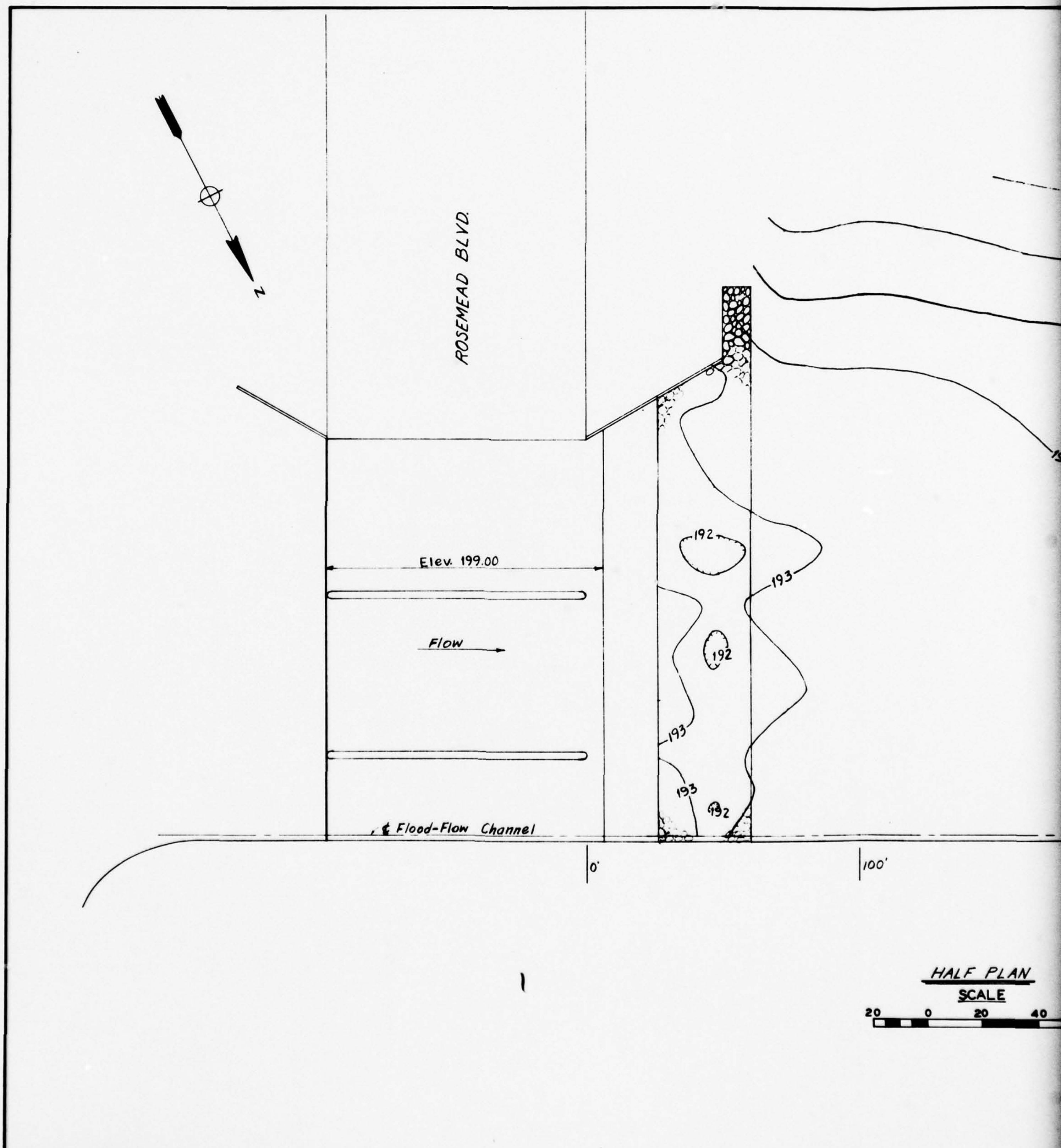
TAILWATER ELEVATION 211.5

WHITTIER NARROWS FLOOD-CONTROL BASIN

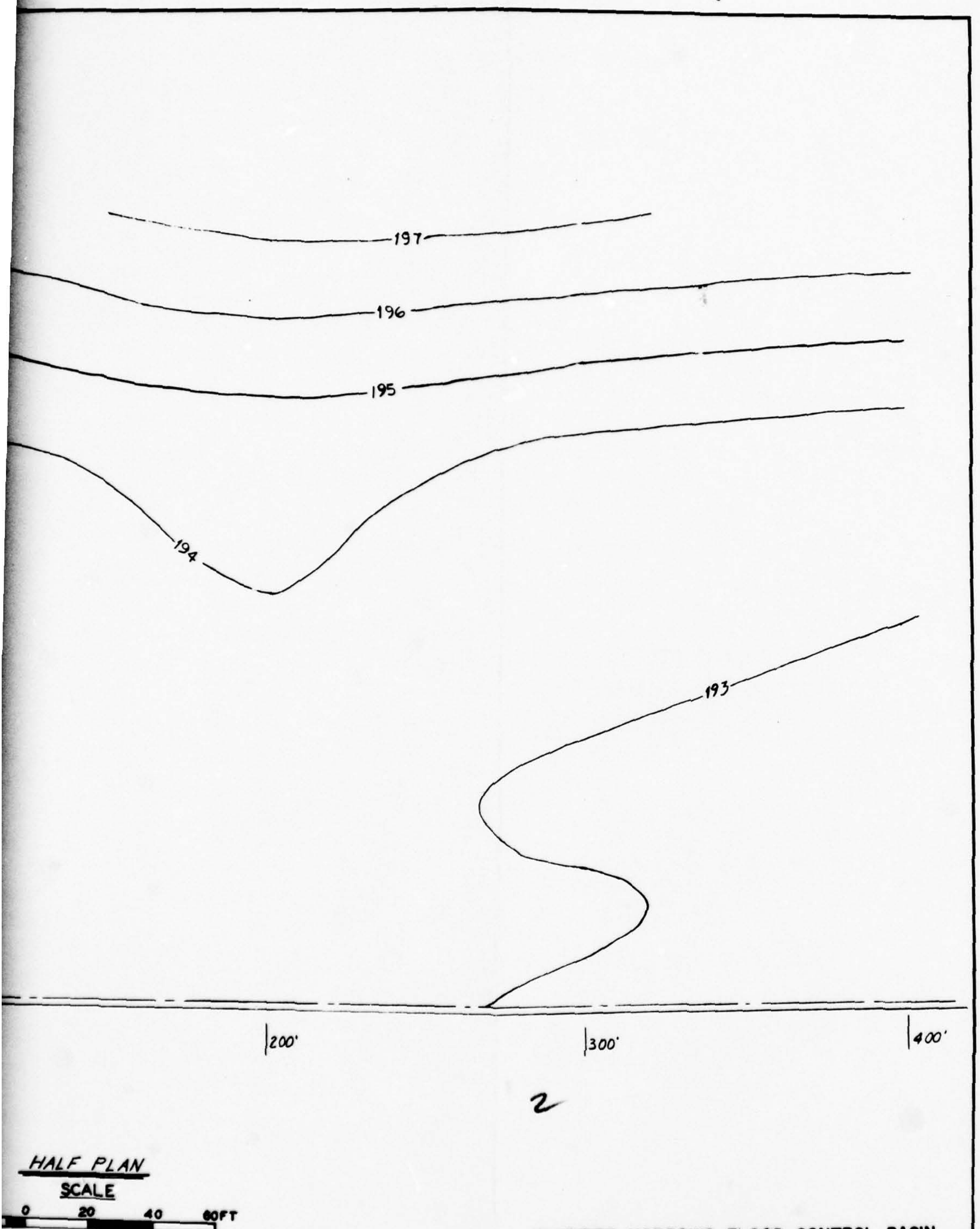
ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN
TEST 9

DISCHARGE = 40,000 CFS, DURATION = 1 HOUR



N
ROSEMEAD BLVD.
Elev. 199.00
Flow
192
192
192
193
193
193
194
℄ Flood-Flow Channel
0'
100'
1
HALF PLAN
SCALE
20
0
20
40



2

DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

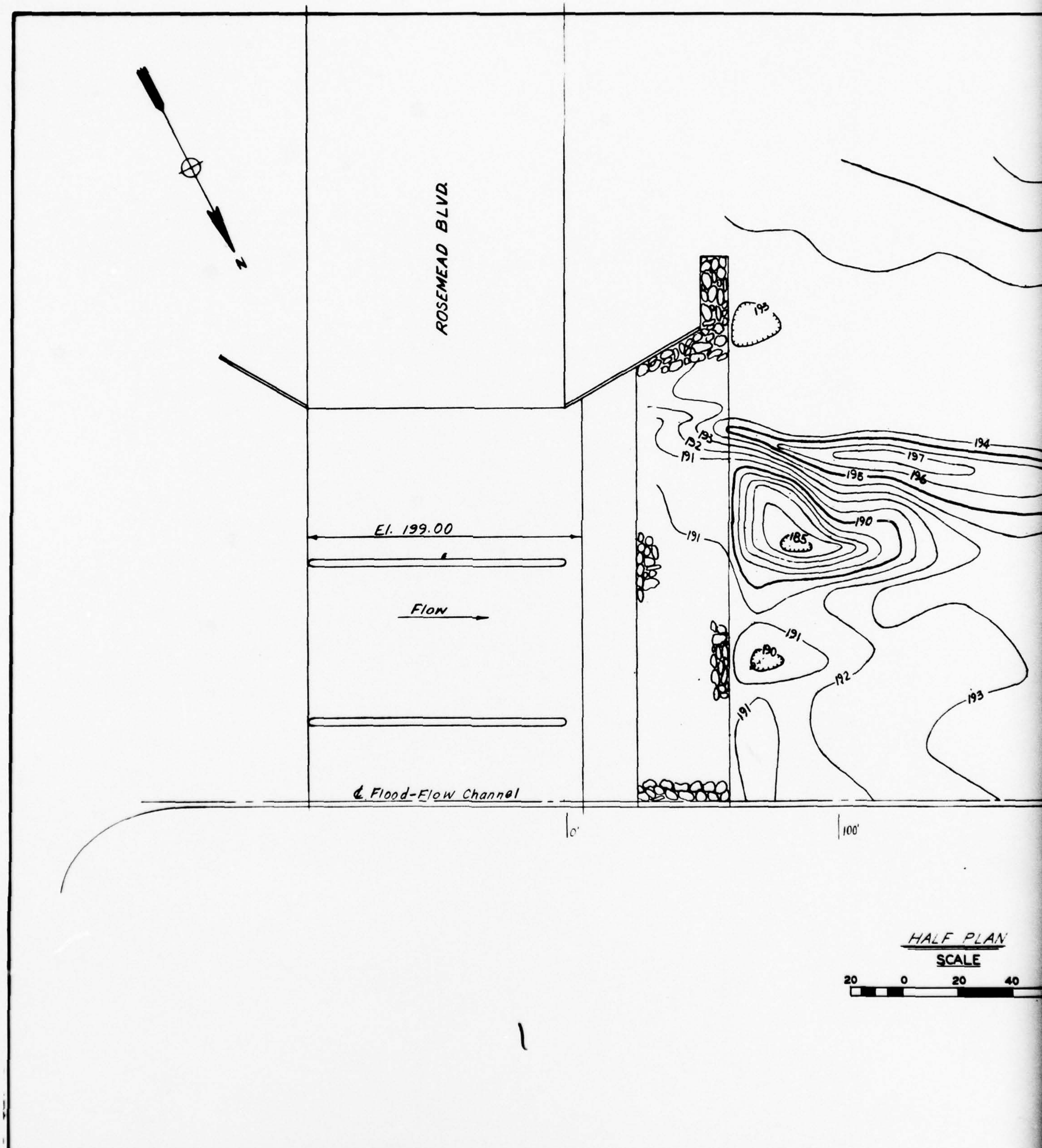
TAILWATER ELEVATION 211.0

WHITTIER NARROWS FLOOD-CONTROL BASIN

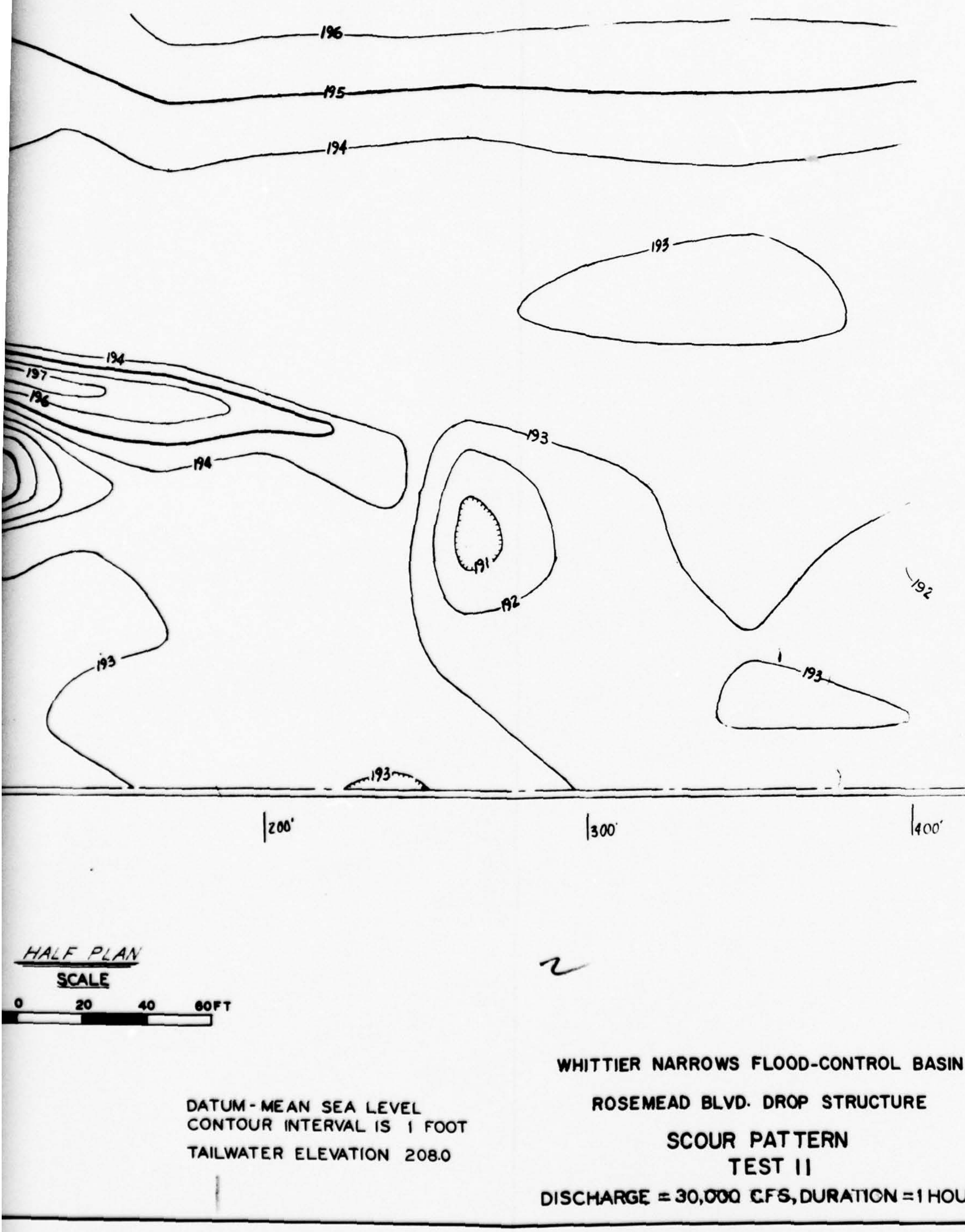
ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN
TEST 10

DISCHARGE = 30,000 CFS, DURATION = 1 HOUR



HALF PLAN
SCALE

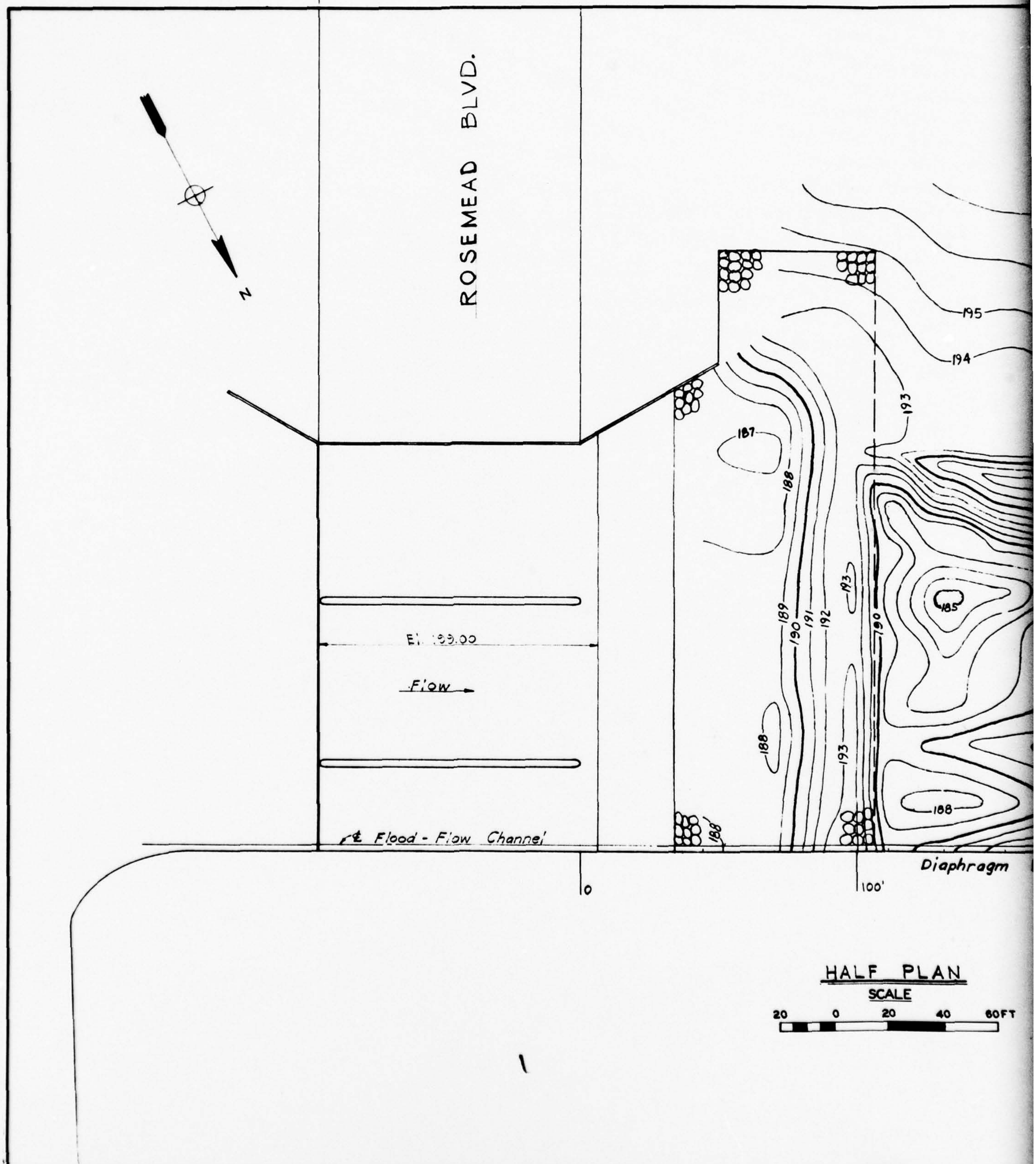


DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT
TAILWATER ELEVATION 208.0

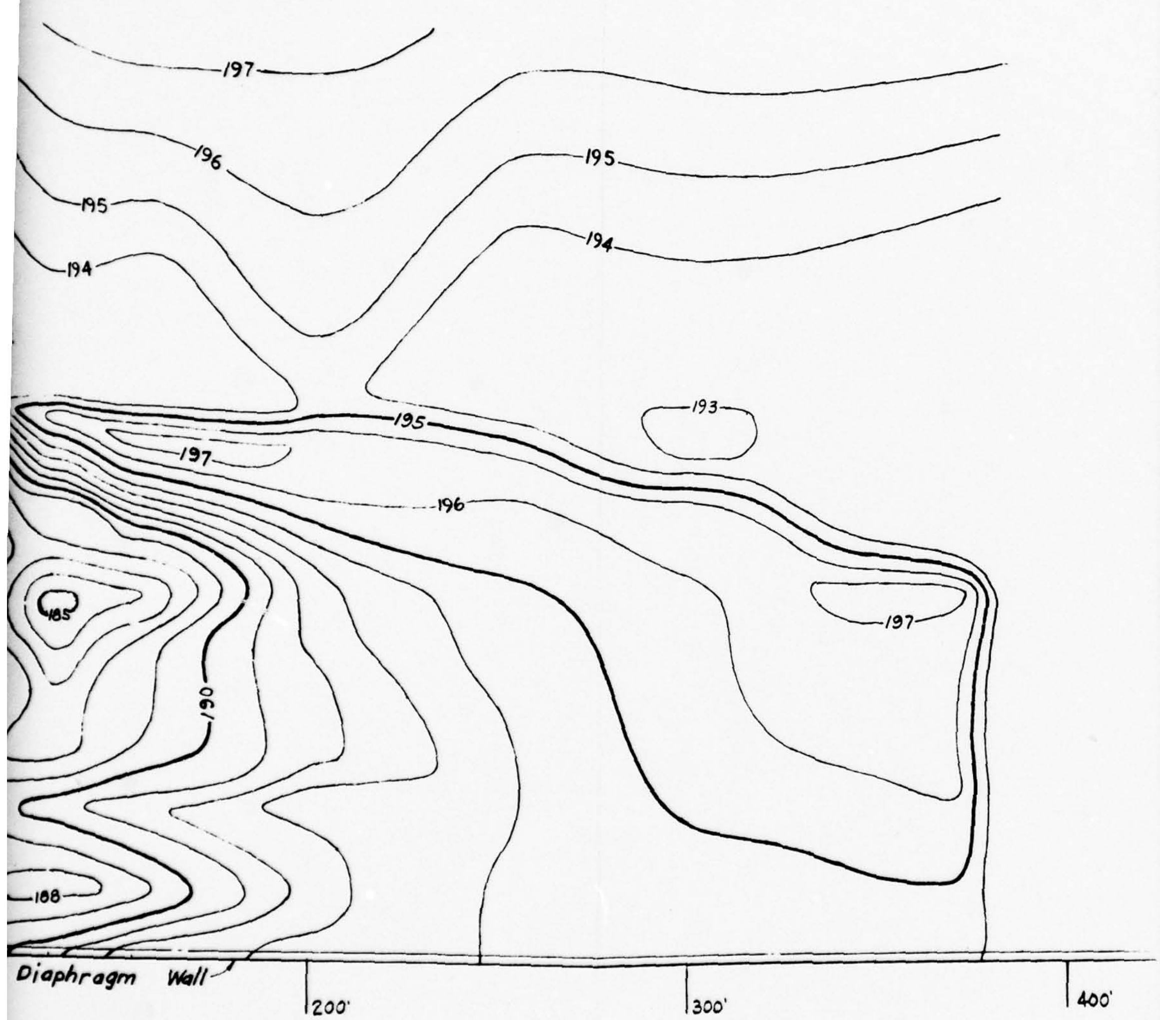
WHITTIER NARROWS FLOOD-CONTROL BASIN

ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN
TEST 11
DISCHARGE = 30,000 CFS, DURATION = 1 HOUR



ROSEMEAD BLVD.
N
El. 199.00
Flow
℄ Flood - Flow Channel
187
188
189
190
191
192
193
194
195
185
188'
0
100'
Diaphragm
HALF PLAN
SCALE
20 0 20 40 60 FT



LAN

40 60FT

DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

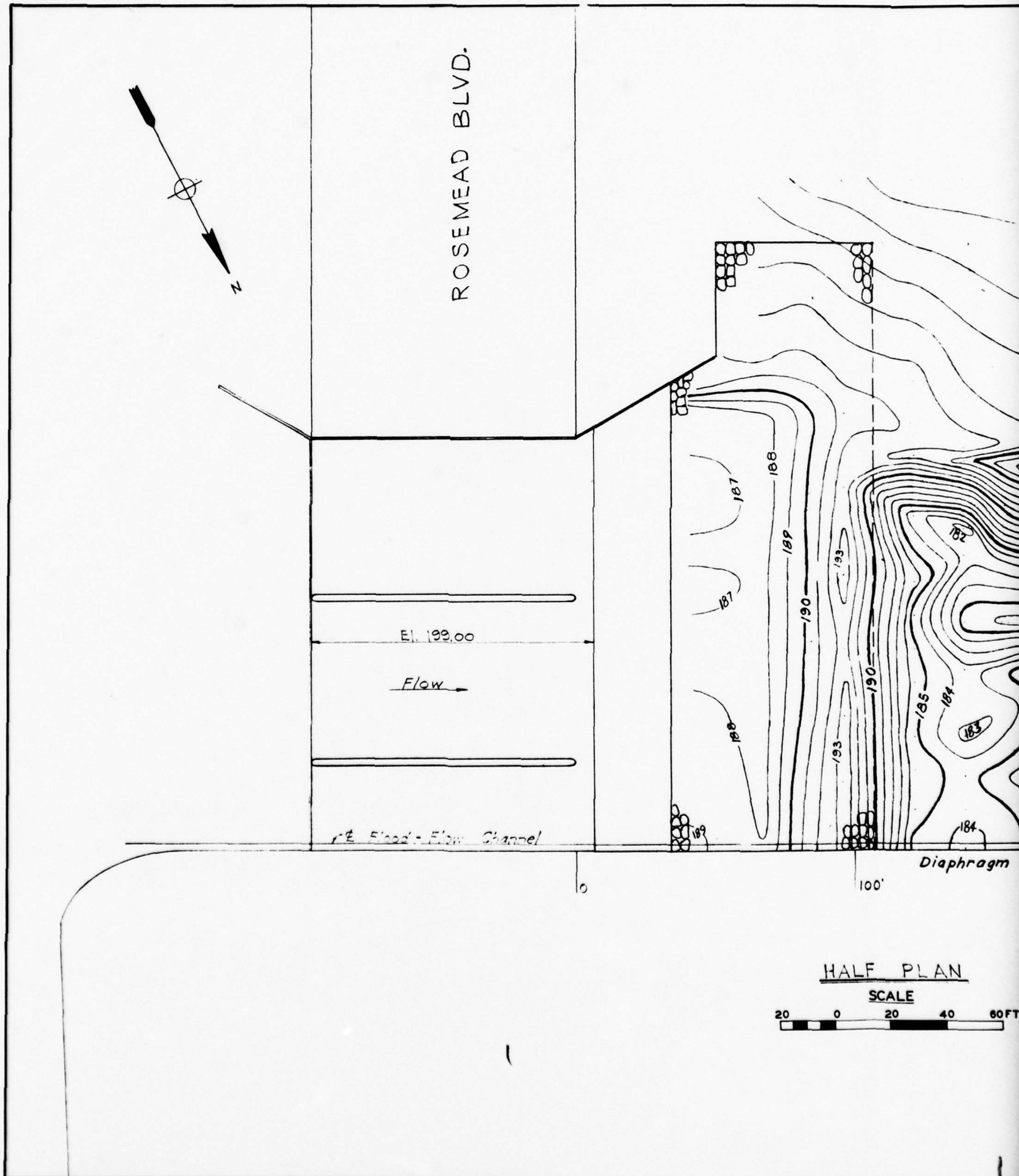
TAILWATER ELEVATION 208.0

WHITTIER NARROWS FLOOD-CONTROL BASIN

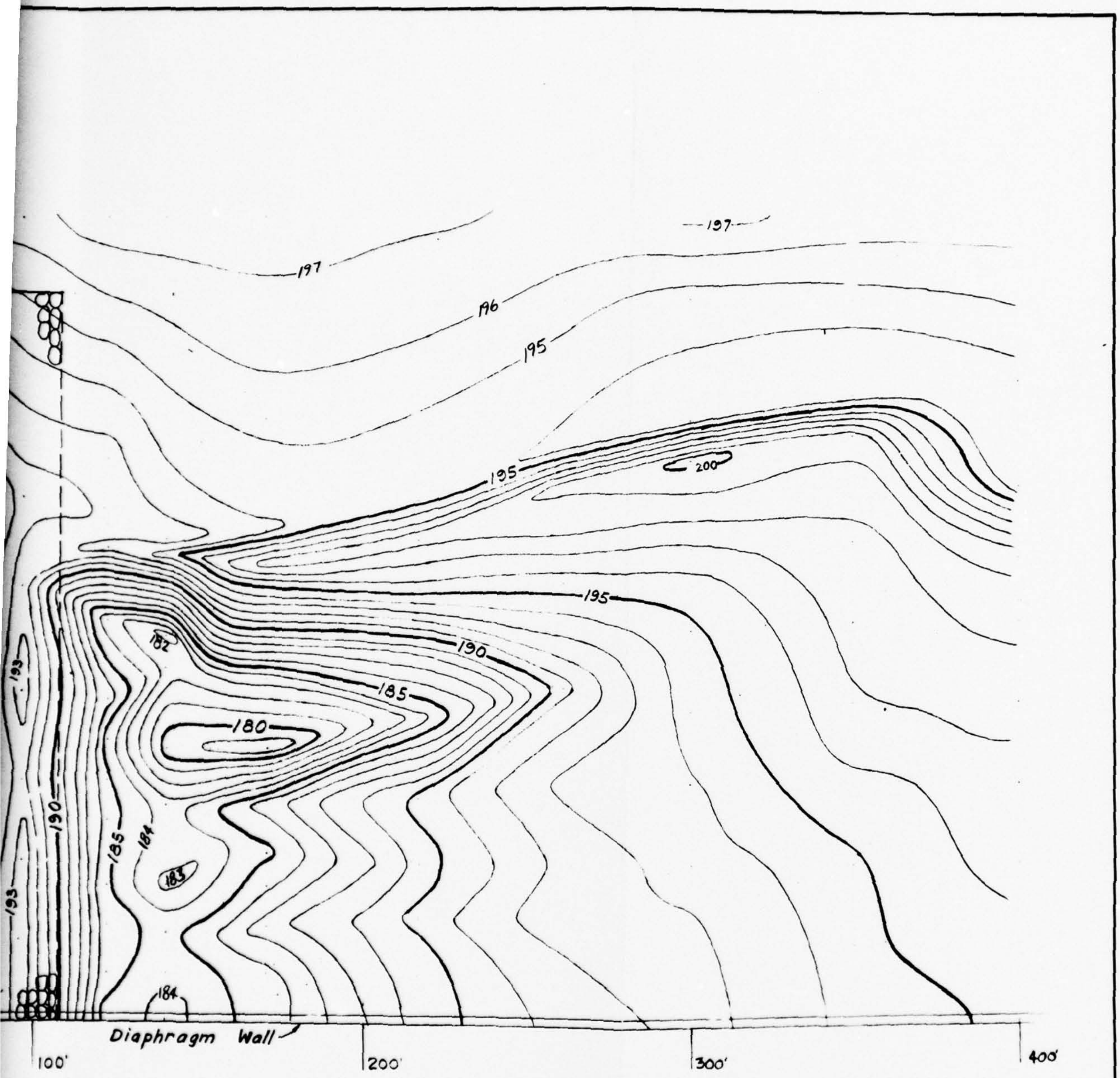
ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN
TEST 12

DISCHARGE = 30,000 CFS, DURATION = 1 HOUR


ROSEMEAD BLVD.
N
El. 199.00
Flow
℄ Flood-Flow Channel
0
100'
Diaphragm
187
188
189
190
193
182
183
184
185
SCALE
20 0 20 40 60FT

HALF PLAN



2

HALF PLAN

SCALE

0 20 40 60 FT

DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

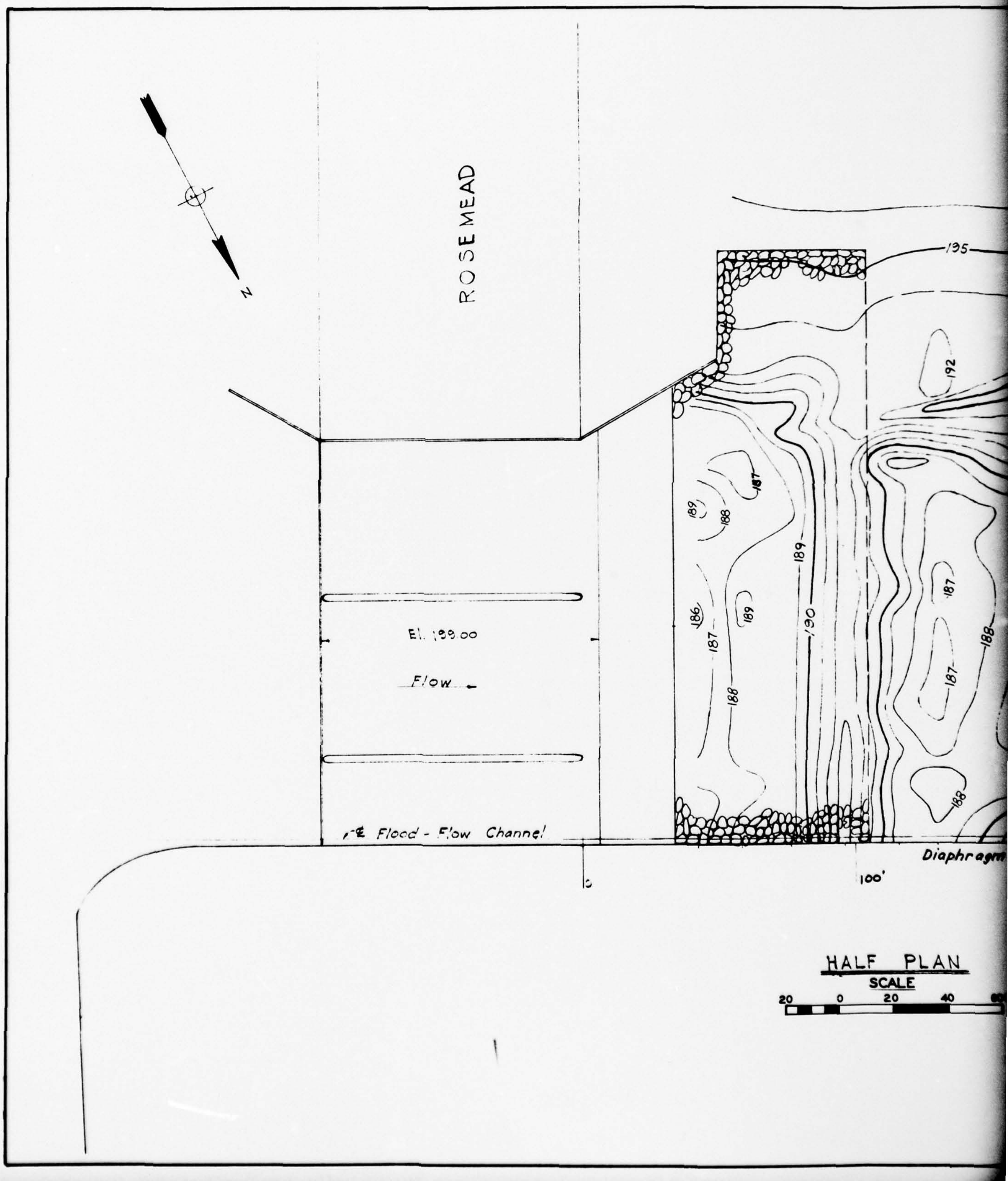
TAILWATER ELEVATION 207.7

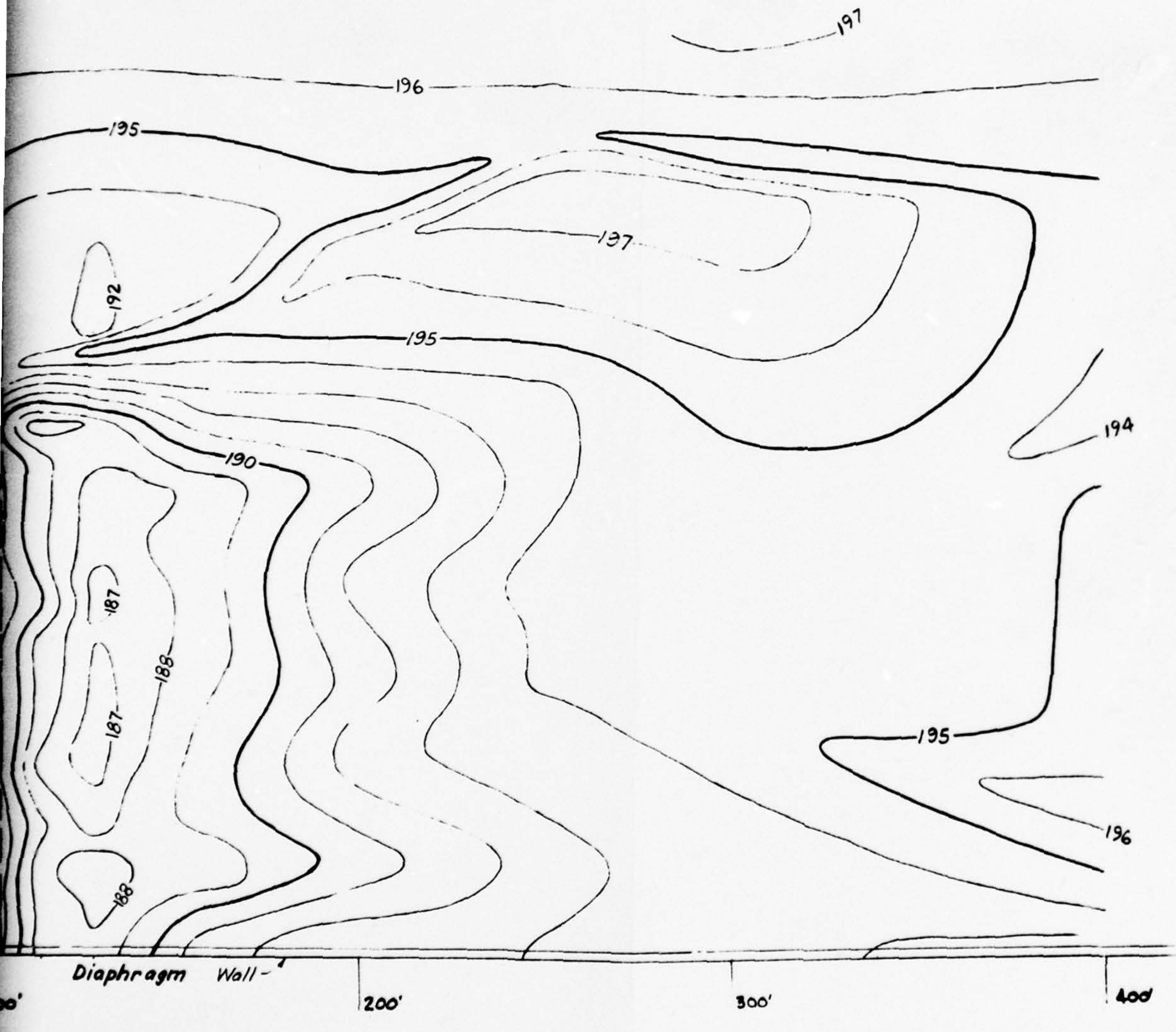
WHITTIER NARROWS FLOOD-CONTROL BASIN

ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN
TEST 13

DISCHARGE = 40,000 CFS, DURATION = 1 HOUR


N
ROSEMEAD
195
192
187
189
188
186
187
188
189
190
189
187
188
187
188
El. 199.00
Flow
℄ Flood - Flow Channel
Diaphragm
0
100'
HALF PLAN
SCALE
20 0 20 40



F PLAN

SCALE

20 40 60FT

DATUM - MEAN SEA LEVEL
CONTOUR INTERVAL IS 1 FOOT

TAILWATER ELEVATION 203.5

WHITTIER NARROWS FLOOD-CONTROL BASIN

ROSEMEAD BLVD. DROP STRUCTURE

SCOUR PATTERN
TEST 14

DISCHARGE = 30,000 CFS, DURATION = 1 HOUR

PART VI: SAN JOSE DIVERSION CHANNEL AT SAN GABRIEL RIVER

The Prototype

71. San Jose Creek rises in the extreme eastern end of the San Gabriel Valley and flows southwesterly. The San Jose Creek normally would flow into the San Gabriel River just downstream of the spillway (see Plate 2), but in order to control flood flows in San Jose Creek it was planned to divert the flows and form a confluence with the San Gabriel River upstream from the termination of the east embankment of Whittier Narrows Dam. The diversion channel is trapezoidal in cross section, with a base width of 150 ft and 1V-on-2.30H side slopes 16 to 17 ft in height. The San Gabriel River is trapezoidal in cross section. The channel upstream from the confluence has a base width of 450 ft; the channel downstream from the confluence has a base width of 500 ft. The channel upstream of the confluence will carry a discharge of 61,000 cfs; at the junction, the combined discharge is 99,000 cfs.

Description of Model

72. The model, constructed to an undistorted scale ratio of 1:60, reproduced 3,200 ft of the San Jose diversion channel and 6,000 ft of San Gabriel River at which 5,000 ft was downstream from the confluence. The model is an integral part of the Whittier Narrows Dam general model. The levees and stabilizers were cast in concrete. The finished concrete levees and the sand streambed closely approximated the assumed design "n" value of 0.030 for the prototype. Crushed rock was used to simulate the quarrystone used as toe stone protection along the levees and rock protection downstream of the stabilizers. The rock used in the model had a specific gravity of 2.6 and was assumed to be equal to that used in the prototype.

Original Design

73. The original design was constructed as shown in Photo 63. The San Jose Creek, upstream from the confluence with the San Grabriel River, is a 150-ft-wide trapezoidal channel designed to carry 38,000 cfs. San Jose Creek enters the junction with a 1,250-ft-radius curve. The San Gabriel River from the junction downstream is 500 ft wide and the design discharge for this main channel is 99,000 cfs. The stabilizers on the San Gabriel River were located at sta 1214+00 and 1200+00. The stabilizer on the San Jose Creek was located at sta 29+85.64 near the semicircular nose of the junction. A rock protection apron 40 ft by 6 ft thick was placed below each stabilizer. A test with the discharge combination of 61,000 cfs in San Gabriel River and 38,000 cfs in San Jose Creek was made for a run of 1-1/2-hr duration. Particular attention was given to (a) scour of the invert along right levee toe of the diversion channel, (b) scour downstream of each stabilizer, and (c) scour of the invert at the confluence with San Gabriel River. After 30 min, rock downstream of the San Jose stabilizer started to move. At the end of 1 hr the right toe of levee below stabilizer at sta 1214+00 became exposed. Observation of flows during the test and of the scour pattern after the run indicated excessive scour along the toe of the right levee in the curved reach of the diversion channel upstream of the stabilizer, in the channel bottom just downstream for the confluence nose, and along the left levee of the San Gabriel River downstream of the confluence. Undercutting developed around the nose and portion of the stabilizer near the confluence. The semicircular nose and stabilizer would have failed completely. The stabilizer located at sta 1214+00 was also damaged from scour where extensive undercutting developed at the right upstream toe of the stabilizer. The scour results of the test are shown in Photo 64. Alterations were limited to the placement of rock in those areas which were subject to scour.

Alternative Design 1

74. This design was similar to the original design except for the

rock protection that was added to the confluence nose and along the toe of right levee of the diversion channel for a distance of 300 ft upstream from the stabilizer. Visual observations were made of the confluence for discharges of 61,000 cfs in San Gabriel River and 38,000 cfs in San Jose diversion channel. Photo 65 shows that the rock placed along the toe of right levee was very effective in reducing the scour that occurs upstream of the San Jose stabilizer. The rock protection kept the levee from being undercut. There was movement of rock downstream of the stabilizers but not as severe as in the previous test; however, the deep scour hole downstream of the confluence nose was similar to the one that developed in the original design test.

Alternative Design 2

75. In this design, the rock protection along the toe of right levee was lengthened to 500 ft and a rock fillet was added to the right upstream edge of the stabilizer. Rock protection also was placed along the toe of right levee for a distance of 250 ft below the stabilizer at sta 1214+00. The rock protection in place before the test run is shown in Photo 66. A model test was conducted with a flow combination of 61,000 cfs in San Gabriel River and 38,000 cfs in San Jose diversion. With this flow combination, backwater from San Gabriel produced an undulating jump that began just downstream of the San Jose stabilizer. Results of the test indicated that in general, the rock protection was satisfactory. The scour pattern of the channel bottom downstream from the confluence extended diagonally across the width of the San Gabriel River for a distance of about 900 ft (Photo 67). A second test was conducted with a discharge of 15,000 cfs in San Gabriel River and the maximum discharge (38,000 cfs) in San Jose diversion. This flow combination would not produce backwater nor induce a hydraulic jump. The flow over the stabilizer impinged on the downstream rock protection causing a deep scour hole downstream of the stabilizer. Most of the downstream rock protection was washed away. Photo 68 shows flow conditions and scour.

Final Design

76. To reduce scouring of the immediate area downstream of the confluence and eliminate the detrimental undercutting of the levees, a new design concept was adopted as shown in Plate 61 and in Photo 69. The right levee of San Jose diversion, which was previously concentric with the left levee, was constructed on a continuous straight alignment to where it joins the left levee of San Gabriel River with a 50-ft-radius nose. The left levee of San Jose diversion and the left levee of San Gabriel River are connected with a 925-ft-radius curve. The San Jose stabilizer was located upstream of the confluence nose at sta 32+85.00. No change was made to the location of the San Gabriel stabilizer; it remained at sta 1214+00. To reduce the scour at the downstream side of the stabilizers as determined in the designs previously tested, rock was placed as shown in Photo 69. The rock apron below the stabilizers was 30 ft long by 6 ft thick. The toe rock along the levee and around the confluence nose had a berm 10 ft wide and 6 ft thick.

77. The first observation and measurements were made with a flow combination of 61,000 cfs in San Gabriel River and 38,000 cfs in San Jose diversion, producing a combined discharge of 99,000 cfs in San Gabriel River downstream from the confluence. When all test data had been obtained with this combination, observations and measurements were made with a flow combination of 15,000 cfs in San Gabriel River and 38,000 cfs in San Jose diversion.

78. With the 61,000- and 38,000-cfs combination, the flow was generally uniform in depth with no disturbances of great magnitude. The jump downstream from the stabilizer was undular in form. Flow conditions and resulting scour for this combination are shown in Photo 70. The scour photographs show that the streambed was uniformly scoured across the entire width of channel downstream of the stabilizer and verify the existence of a uniform flow distribution downstream of the San Jose stabilizer. The scour photographs indicate that there was very little movement of the rock and no undercutting of the rock apron.

Examination of the levees indicated that negligible scour occurred. The scour pattern is shown in Plate 61.

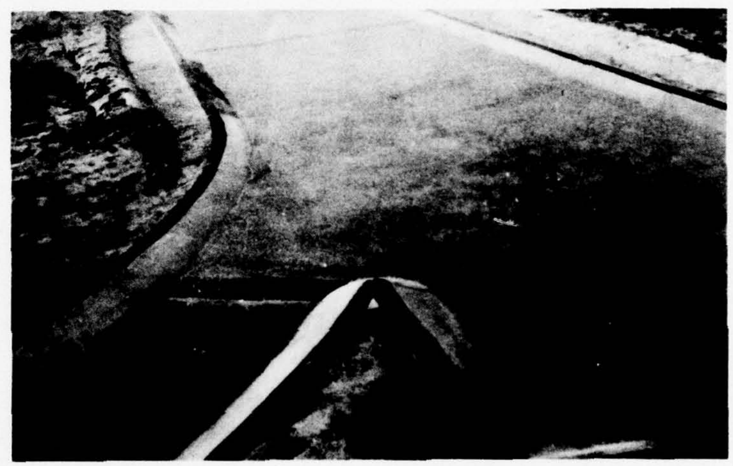
79. The simultaneous flows of 15,000 cfs in San Gabriel River and 38,000 cfs in San Jose diversion produced a marked increase in scour. This indicates that the backwater produced by the 61,000 cfs in San Gabriel River in the previous flow combination was effective in minimizing the scour which would develop downstream of the stabilizer and channel downstream. Photo 71 shows the scour after a 1-1/2-hr run; the scour pattern is shown in Plate 62. The stabilizers were not scoured; the rock apron provided satisfactory protection for the stabilizers.

80. The last test was to determine if eliminating the toe rock protection along the levee and nose and lengthening the rock apron to 40 ft would cause undesirable scour. The flow combination of 61,000 cfs in San Gabriel River and 38,000 cfs in San Jose diversion was used in the test. There was an increase in depth of scour along the levee toe and the confluence nose although there was no marked change in the scour pattern. No photographs were taken but depth of scour was measured and plotted; scour pattern is shown in Plate 63.

81. Results of the tests indicated that in general, the design of the confluence was satisfactory. No flow disturbances developed in San Gabriel River downstream from the confluence and freeboard on the levees provided in the design was adequate. Scour tests of the streambed and the simulation of quarrystone used as toe stone protection for the levee and rock apron below the stabilizers were considered to be qualitative; but the flow conditions observed in the model indicated that the quarrystone would be satisfactory as rock protection for the stabilizers and levees.

Looking upstream

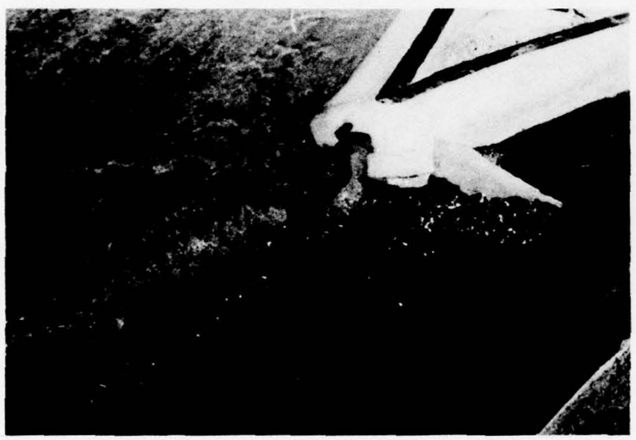
Looking upstream



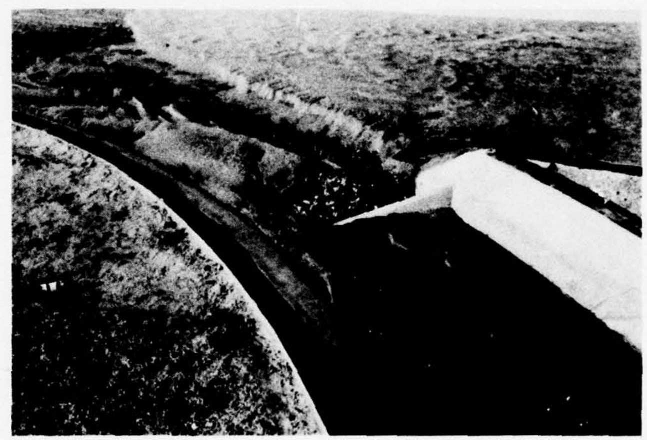
Looking downstream

Photo 63. Original design, San Jose diversion channel

Looking upstream



Looking upstream (close-up)



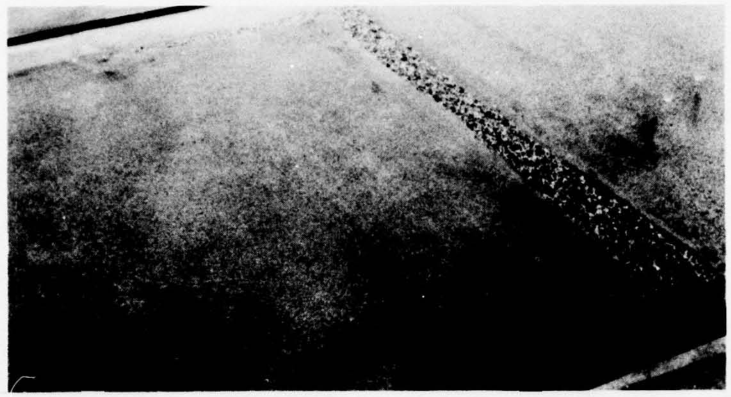
Looking downstream

Photo 64. Original design, scour after 1-1/2-hr run; discharge San Gabriel River 61,000 cfs, San Jose diversion 38,000 cfs

Looking downstream

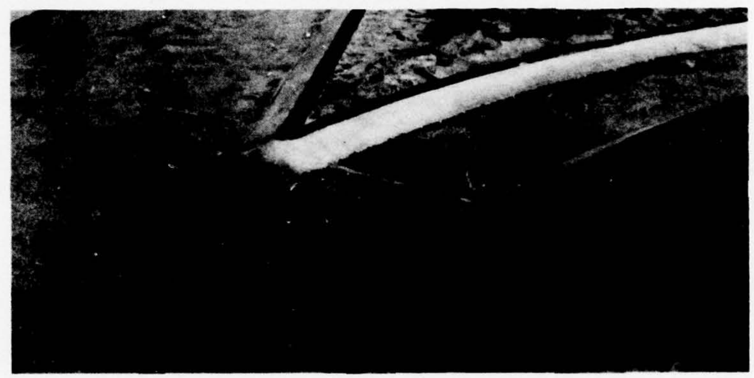
Looking upstream

Photo 65. Alternative design 1, scour after 1-1/2-hr run; discharge San Gabriel River 61,000 cfs, San Jose diversion 38,000 cfs



San Gabriel stabilizer

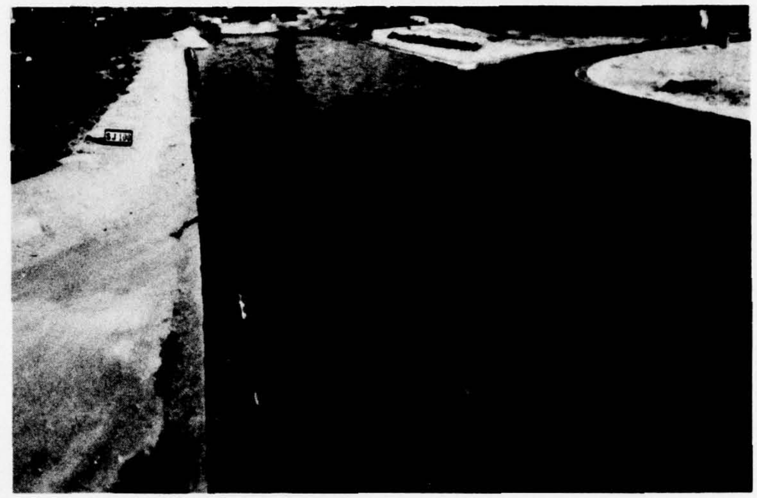
Looking downstream, San Jose stabilizer



Looking upstream, San Jose stabilizer

Photo 66. Alternative design 2, rock protection in place

Looking downstream



Looking upstream

Photo 67. Alternative design 2, flow conditions and scour; discharge San Gabriel River 61,000 cfs, San Jose diversion 38,000 cfs (sheet 1 of 2)

Looking upstream, San Jose stabilizer

Looking downstream, San Jose stabilizer

Looking upstream, San Gabriel stabilizer

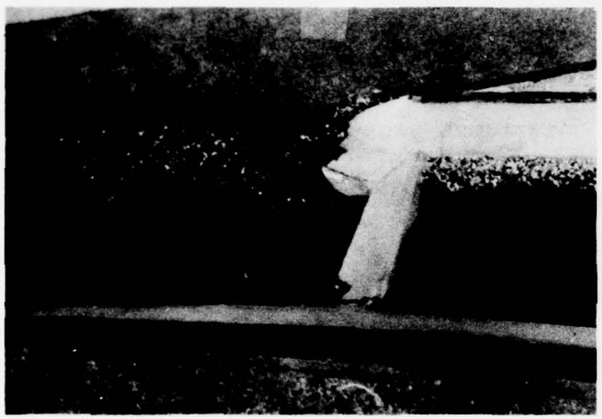
Looking across channel from left side,
San Gabriel stabilizer

Scour after 1-1/2-hr run

Photo 67 (sheet 2 of 2)

Looking downstream

Looking across confluence from left side

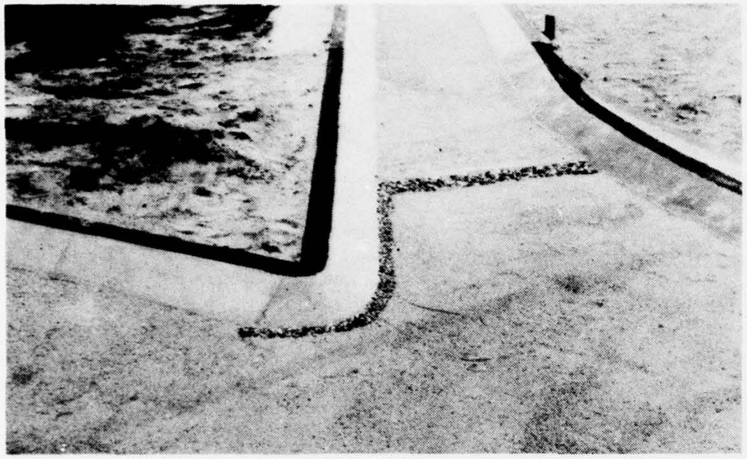


Looking across San Jose stabilizer
from left side

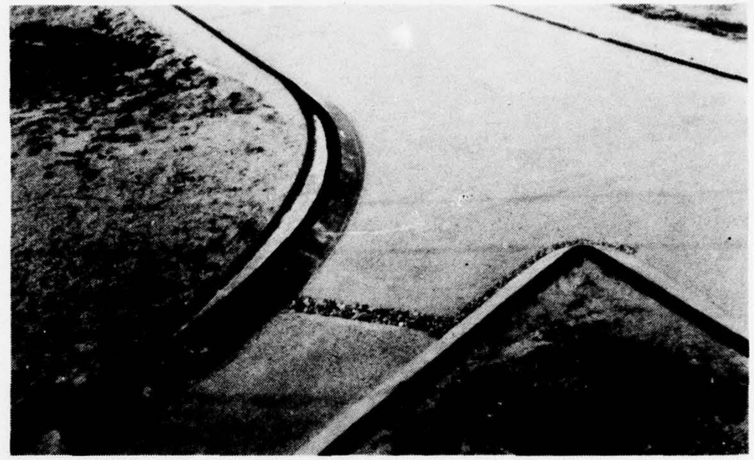
Looking upstream at San Jose stabilizer

Scour after 1-1/2-hr run

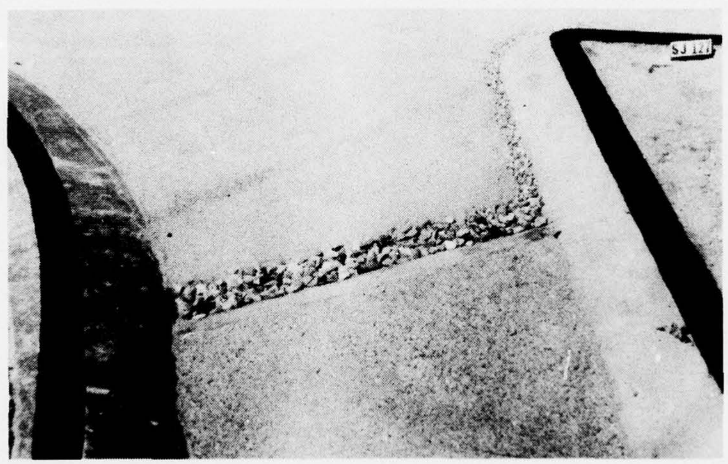
Photo 68. Alternative design 2, flow conditions and scour; discharge San Gabriel River 15,000 cfs, San Jose diversion 38,000 cfs



Looking upstream

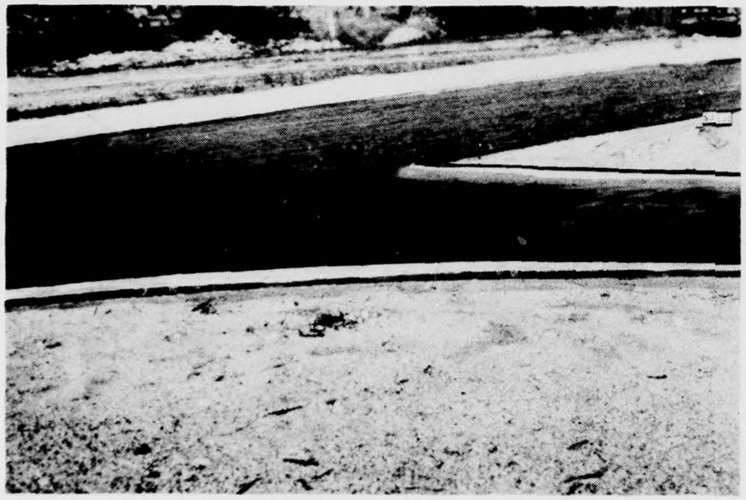


Looking downstream



Looking downstream (close-up)

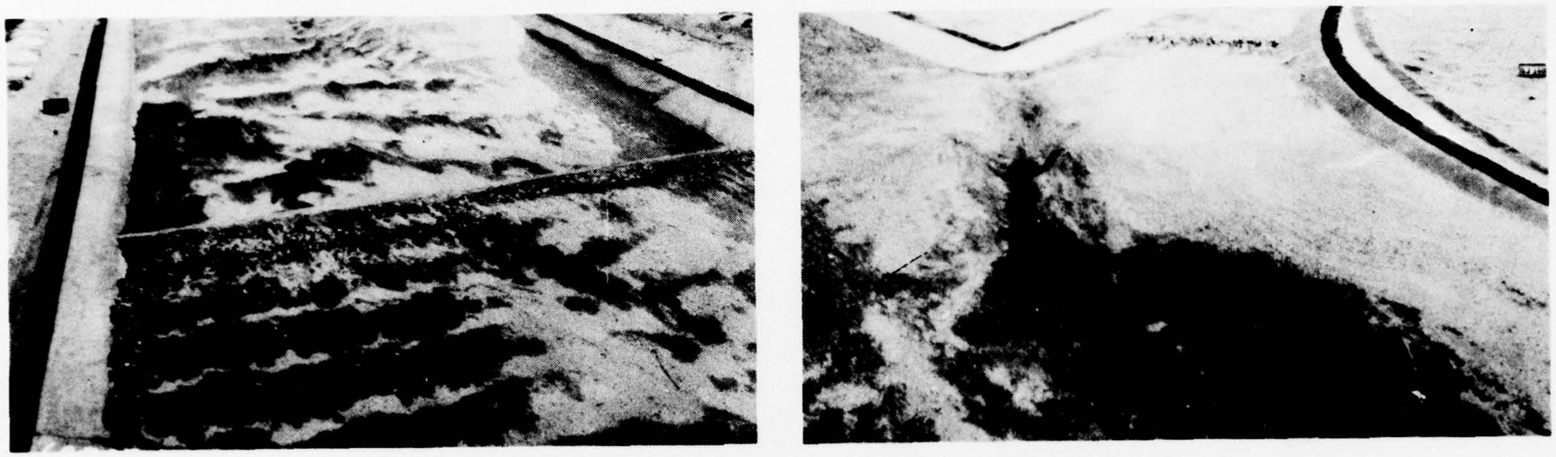
Photo 69. Final design for San Jose diversion channel with rock protection added



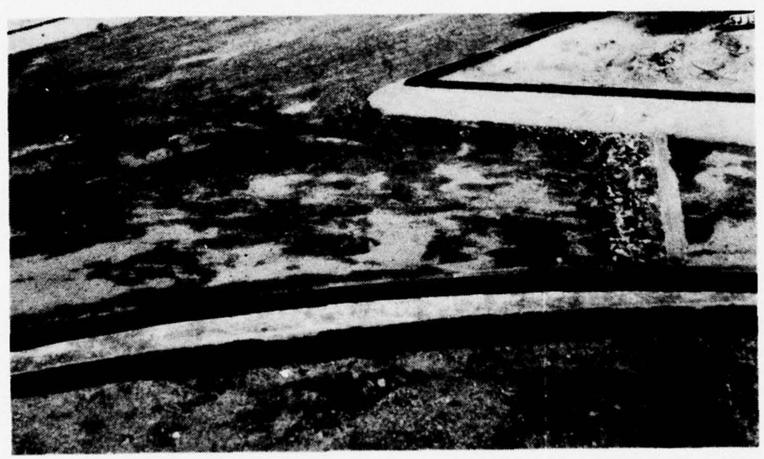
Looking normal to the flow from left side

Looking downstream

Photo 70. Final design for San Jose diversion channel; discharge San Gabriel River 61,000 cfs, San Jose diversion 38,000 cfs (sheet 1 of 2)



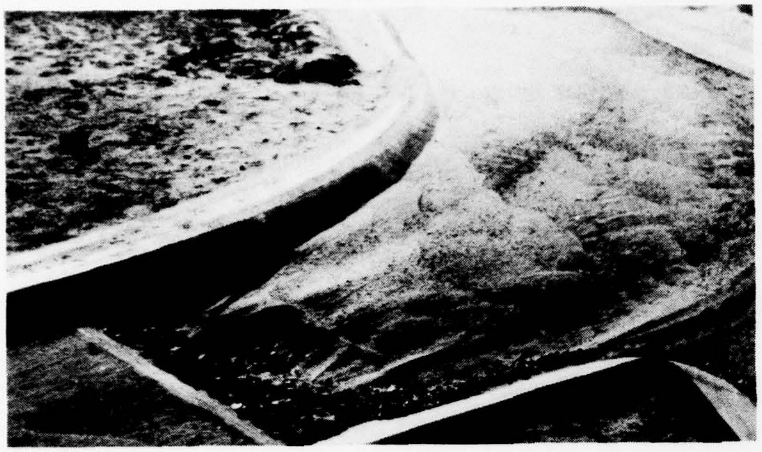
Looking upstream



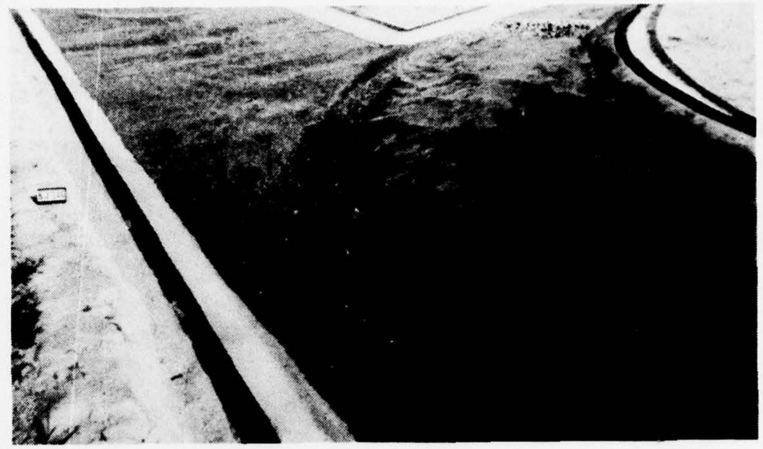
Looking across at confluence, from left side

Scour after 1-1/2-hr run

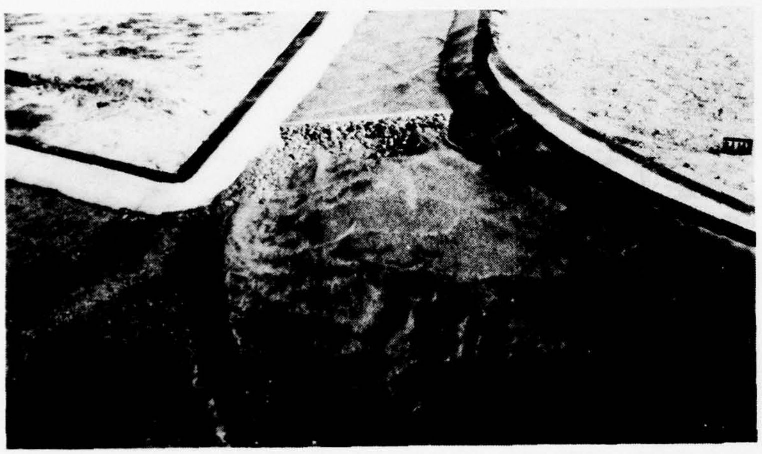
Photo 70 (sheet 2 of 2)

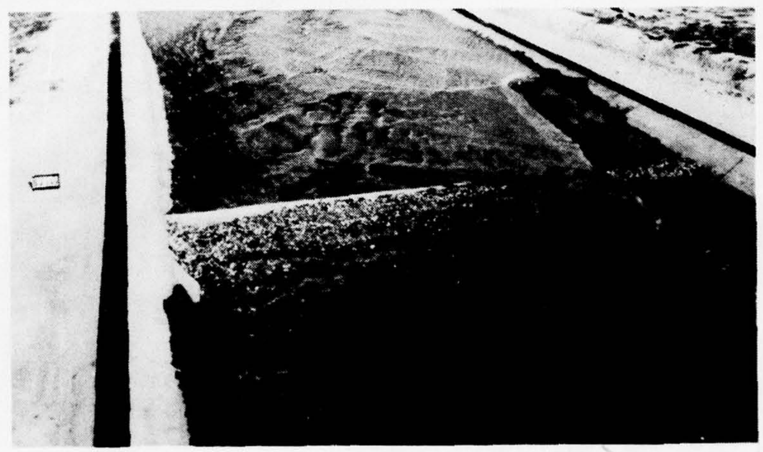


Looking downstream



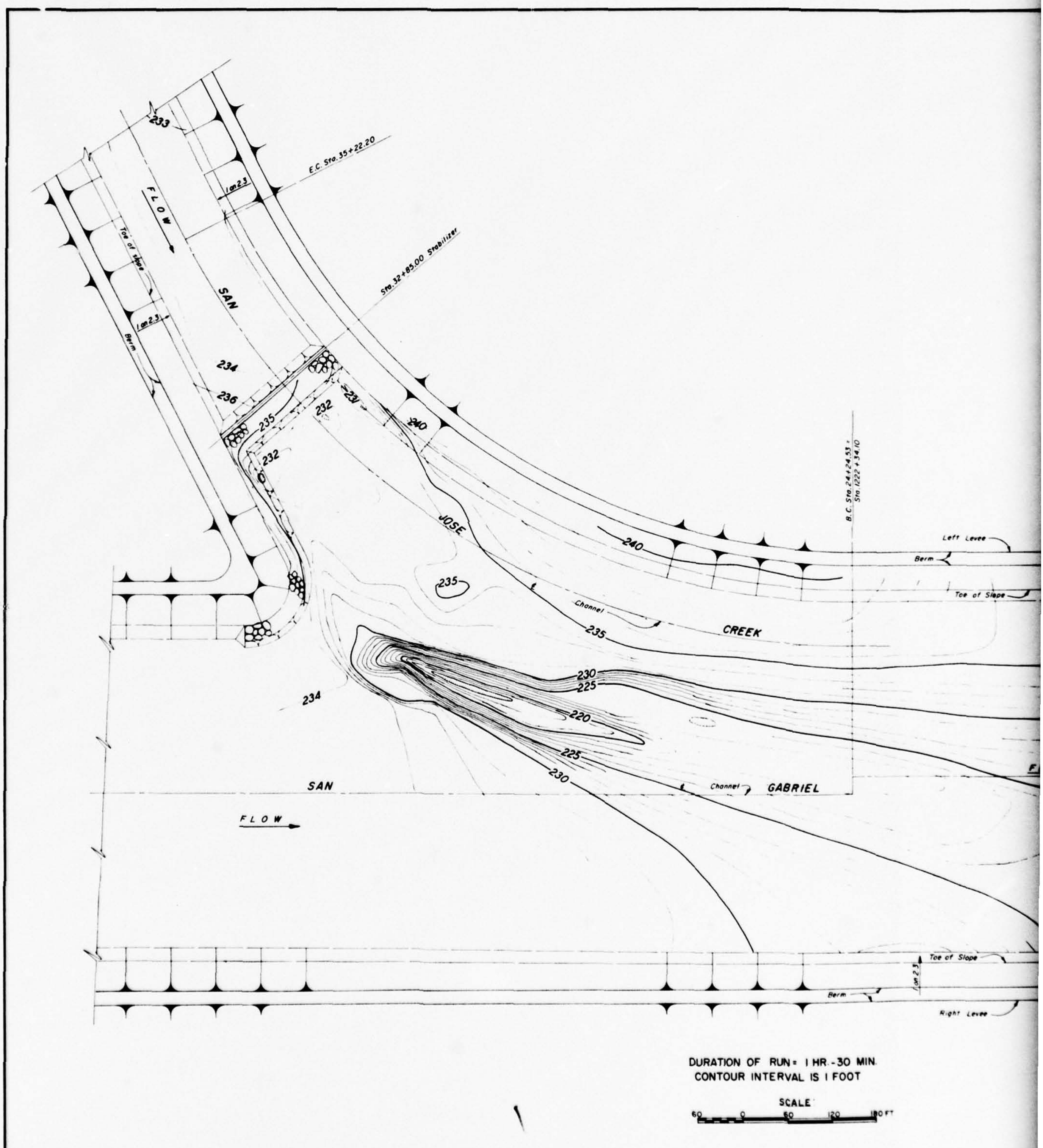
Looking upstream



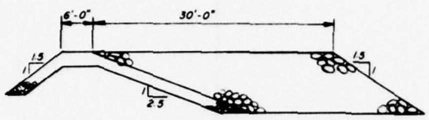


Looking upstream

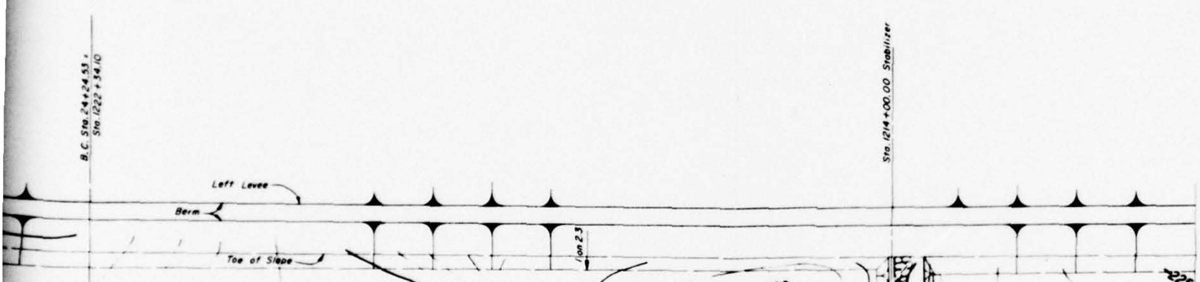
Photo 71. Final design for San Jose diversion channel, scour after 1-1/2-hr run; discharge San Gabriel River 15,000 cfs, San Jose diversion 38,000 cfs



233
E.C. Sta. 35+22.20
1 on 2.3
FLOW
Toe of Slope
SAN
Sta. 32+85.00 Stabilizer
1 on 2.3
Berm
234
236
235
232
231
240
232
JOSE
240
235
Channel
235
CREEK
B.C. Sta. 24+24.53 = Sta. 1222+34.10
Left Levee
Berm
Toe of Slope
230
225
220
225
230
234
SAN
Channel
GABRIEL
FLOW
Toe of Slope
1 on 2.3
Berm
Right Levee
DURATION OF RUN = 1 HR.-30 MIN.
CONTOUR INTERVAL IS 1 FOOT
SCALE
60 0 60 120 180 FT



TYPICAL SECTION-STABILIZER
NOT TO SCALE



= 1 HR.-30 MIN.
AL IS 1 FOOT

ALE:
120 180 FT

WHITTIER NARROWS FLOOD-CONTROL BASIN

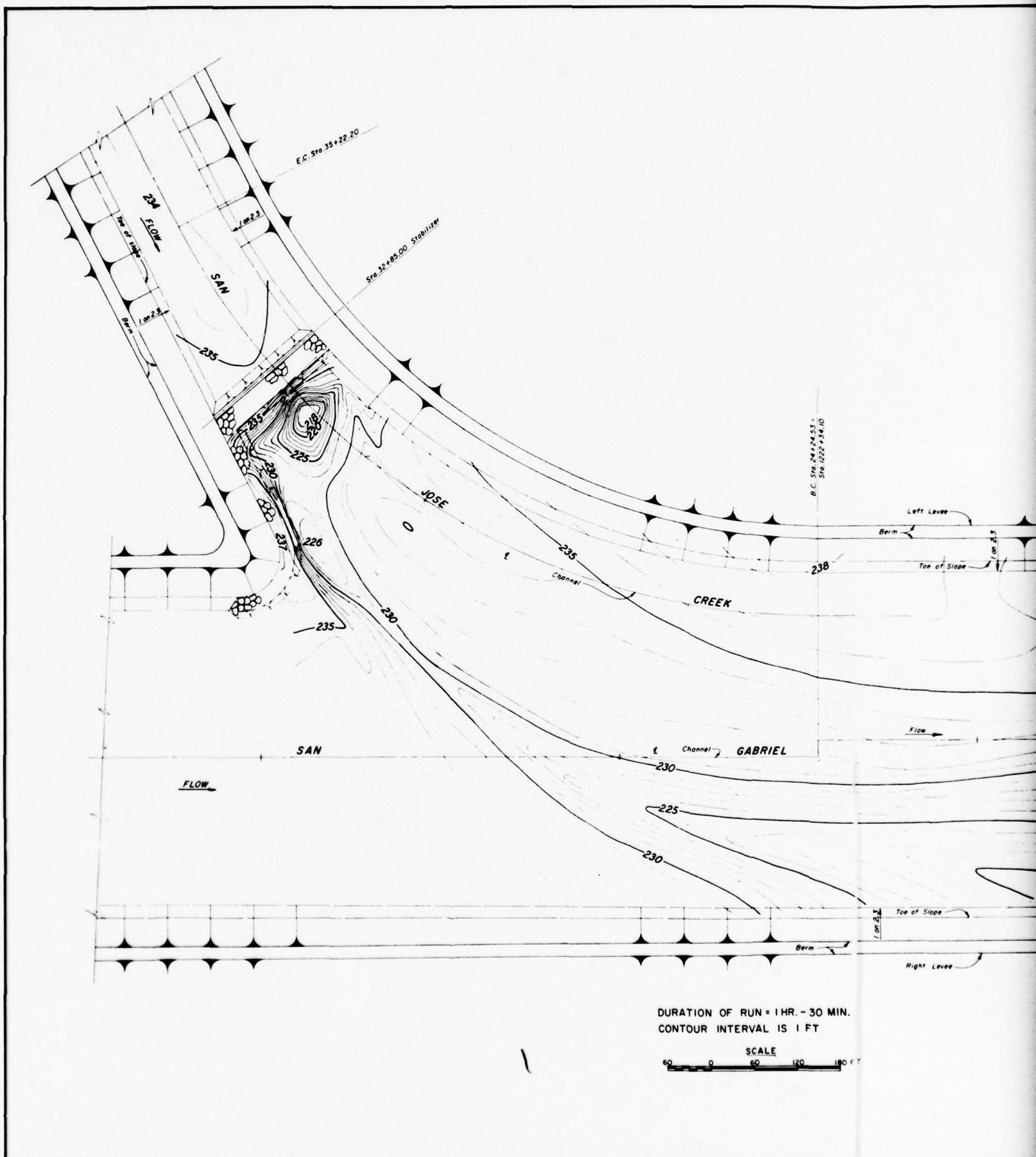
SAN JOSE CREEK DIVERSION CHANNEL

SCOUR PATTERN

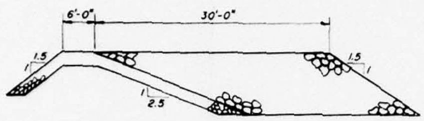
30-FT-WIDE STABILIZER

DISCHARGES: SAN JOSE CREEK 38,000 CFS
SAN GABRIEL RIVER 61,000 CFS

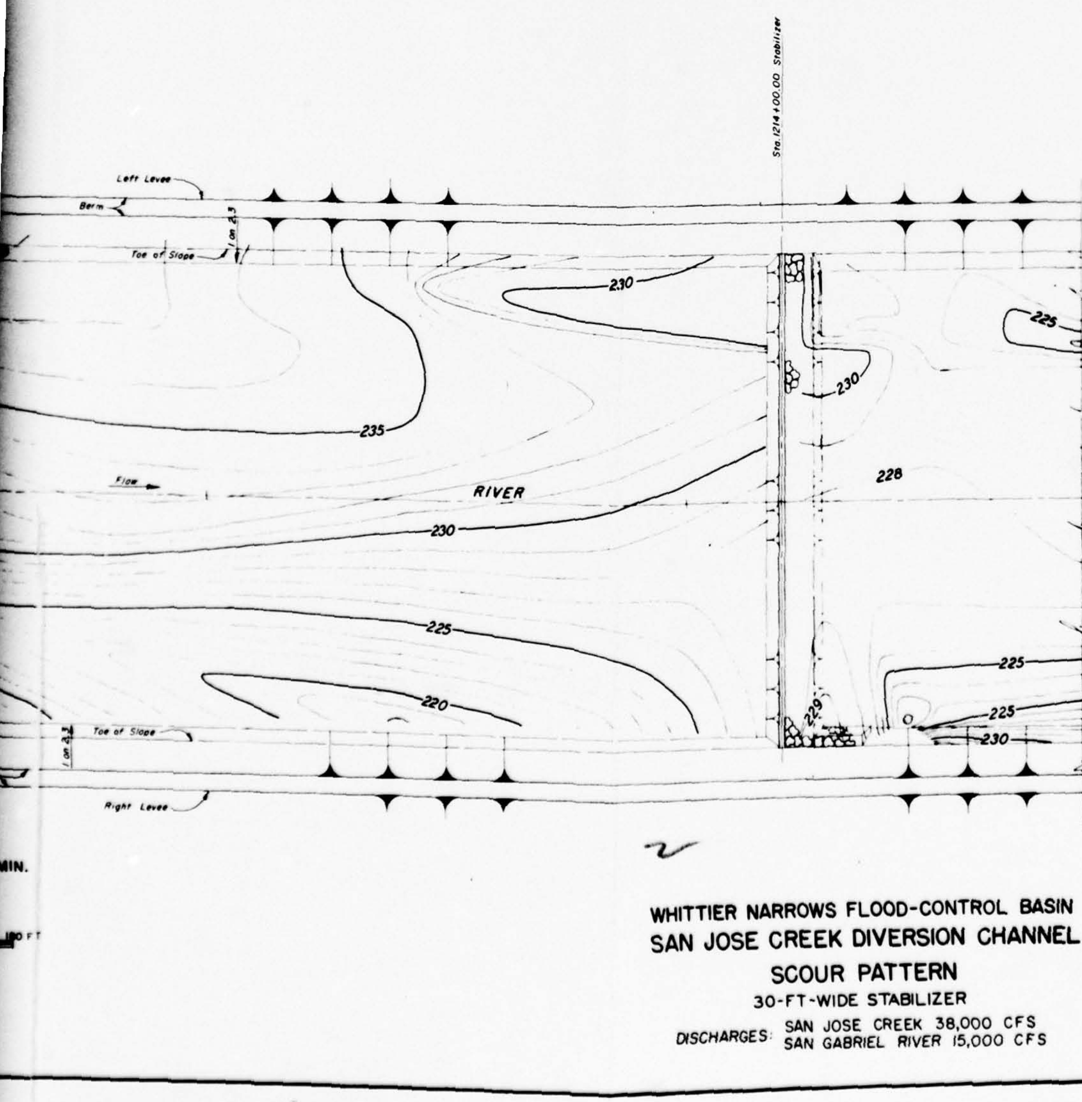
PLATE 61



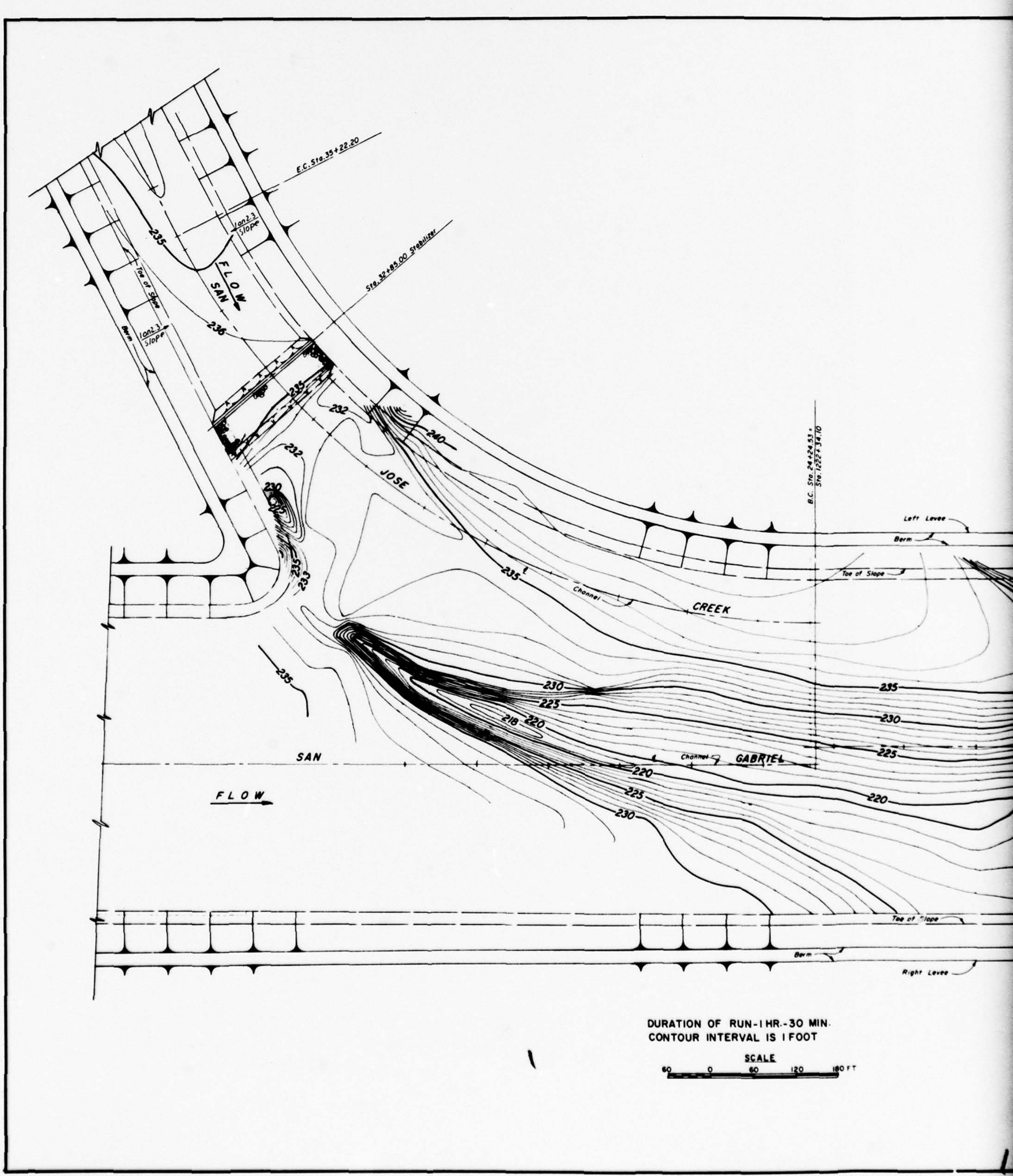
E.C. Sta. 35+22.20
Sta. 32+85.00 Stabilizer
234
FLOW
Toe of slope
1 on 2.3
SAN
Berm
235
JOSE
B.C. Sta. 24+24.53 = Sta. 1222+34.10
Left Levee
Berm
Toe of Slope
238
Channel
CREEK
230
225
226
237
SAN
Channel
GABRIEL
FLOW
Flow
Toe of Slope
Berm
Right Levee
DURATION OF RUN = 1 HR. - 30 MIN.
CONTOUR INTERVAL IS 1 FT
SCALE
60 0 60 120 180 FT



TYPICAL SECTION - STABILIZER
NOT TO SCALE

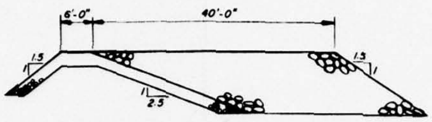


WHITTIER NARROWS FLOOD-CONTROL BASIN
SAN JOSE CREEK DIVERSION CHANNEL
SCOUR PATTERN
30-FT-WIDE STABILIZER
DISCHARGES: SAN JOSE CREEK 38,000 CFS
SAN GABRIEL RIVER 15,000 CFS



DURATION OF RUN-1 HR.-30 MIN.
CONTOUR INTERVAL IS 1 FOOT

SCALE
60 0 60 120 180 FT



TYPICAL SECTION—STABILIZER
NOT TO SCALE

WHITTIER NARROWS FLOOD-CONTROL BASIN
SAN JOSE CREEK DIVERSION CHANNEL
SCOUR PATTERN
40-FT-WIDE STABILIZER
DISCHARGES: SAN JOSE CREEK 38,000 CFS
SAN GABRIEL RIVER 61,000 CFS

PART VII: RIO HONDO INLET CHANNEL INTO WHITTIER NARROWS BASIN

The Prototype

82. The Rio Hondo inlet is concrete-lined, is trapezoidal in cross section with a base width of 150 ft, and has side slopes of 1V on 2.25H and an invert grade of 0.002363. The inlet channel was designed for a discharge of 51,000 cfs.

The Model

83. The model, constructed to a scale ratio of 1:50, reproduced 450 ft of channel and 1,000 ft of the reservoir area. The material used in constructing the model consisted of wood, sand, and rock. Derrick stone was simulated by 1-in. crushed rock. The features that were considered in the study were the channel terminus and the scour of the sand bed downstream of the inlet channel which is located within the reservoir area. A control gate at the forebay was used to obtain the required depth at the upstream end of the channel. The desired reservoir tailwater elevation was maintained by an adjustable tailgate at the downstream end of the model.

Scale Relations

84. The general relations for the transfer of model data to prototype equivalents or vice versa are presented in the following tabulation:

Dimension	Ratio	Scale Relations
Length	L_r	1:50
Area	$A_r = L_r^2$	1:2,500
Velocity	$V_r = L_r^{1/2}$	1:7.07

Dimension	Ratio	Scale Relations
Discharge	$Q_r = L_r^{5/2}$	1:17,678
Roughness	$N_r = L_r^{1/6}$	1:1.919

General Procedure

85. The combined peak inflow to the Whittier Narrows flood-control basin during the reservoir design flood would be 70,000 cfs and the maximum pool elevation developed would be 229.0. The peak inflow from the Rio Hondo inlet channel was assumed to be 29,900 cfs. A design flood hydrograph was used to determine the time-discharge relations for the test runs in the model. The tests were made over a 12-hr (prototype) hydrograph starting with a low discharge of 9,100 cfs and terminating with a discharge of 10,800 cfs on the descending stage of the hydrograph. The design discharge that was used in the design of Rio Hondo inlet channel was 51,000 cfs. With this discharge, the test run was for a 1/2-hr (model) duration and reservoir pool assumed at el 220.0.

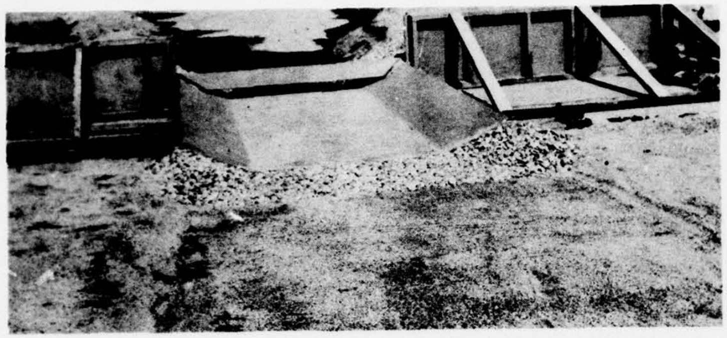
The Original Design

86. The original design was constructed as shown in Plate 64 and Photo 72. The end of the concrete-lined channel was terminated at sta 590+00. A 100-ft long transition composed of ungrouted derrick stone (2,000 lb maximum) expanded the channel from a base width of 150 ft to a base width of 300 ft. The first test was conducted using the design-flood hydrograph having a peak discharge of 29,900 cfs and maximum tailwater elevation of 229.0. There was sufficient tailwater to produce a hydraulic jump for all flows. The hydraulic action of this design with a peak discharge of 29,900 cfs is indicated in Photo 73. The ungrouted derrick stone section demonstrated that satisfactory flow conditions were obtained for a discharge of 29,900 cfs. The stone roughness prevented the acceleration of the flow downstream. Plate 64 and Photo 74 show the relative scour that occurred downstream of the

stone section. With a discharge of 51,000 cfs, and pool elevation of 220.0, the scour was more severe as shown in Photo 75. The lack of sufficient pool depth to produce tailwater resulted in excessive scouring at the downstream end of the inlet channel.

The Final Design

87. In the final design the 150-ft-wide concrete channel was extended downstream 40 ft and rounded ends were added to the levees downstream. Two tests were made on the final design. The first test was conducted using 1/2-in. crushed rock which simulated 700-lb derrick stone for the downstream end protection. The rock protection was placed at a 1V-on-5H slope beginning at the end of the concrete channel and continuing downstream for 50 ft to a depth of 10 ft. The placement of the rock protection is shown in Photo 76. The test was conducted using the same hydrograph that was used in testing the original design. Results of the test with a peak discharge of 29,000 cfs show that most of the rock protection was scoured away (Photo 77). The area scoured during the test is shown in Plate 65. The second test was conducted with 1-in. rock (simulating 2,000-lb maximum derrick stone) for the downstream end protection (Photo 78). The model was tested for a design discharge of 51,000 cfs. Flow conditions and results of this test are shown in Photo 79 and in Plate 66. The 1-in. rock provided satisfactory protection for the end of the concrete channel; therefore this design was considered satisfactory.

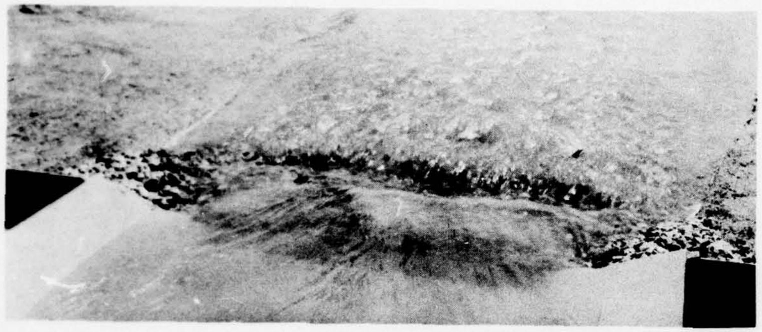


Looking upstream

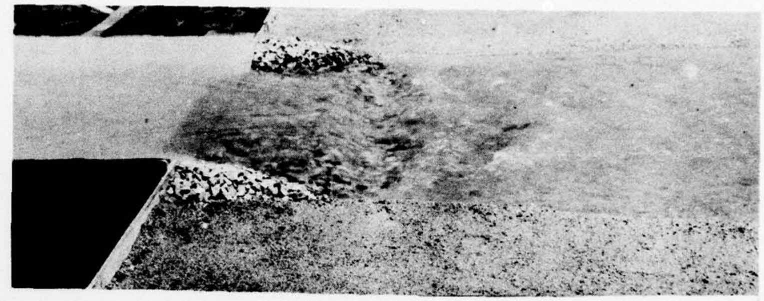
Looking across channel from right side

Looking upstream from left side

Photo 72. Original design, Rio Hondo inlet channel



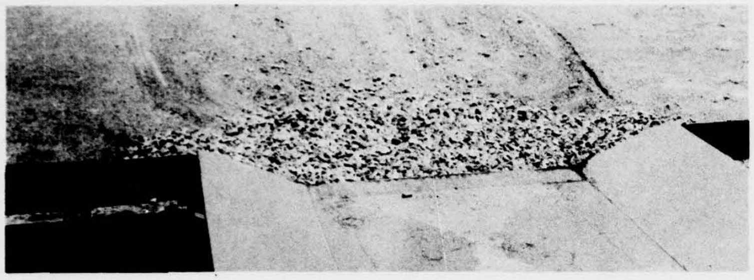
Looking downstream



Looking across channel from right side

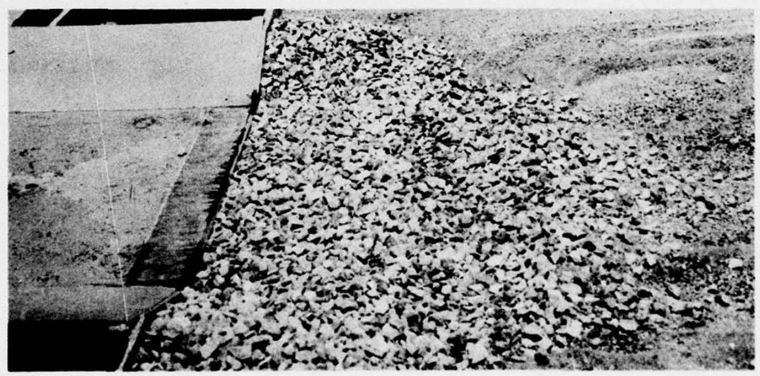
Closeup view from right side

Photo 73. Original design, discharge 29,900 cfs



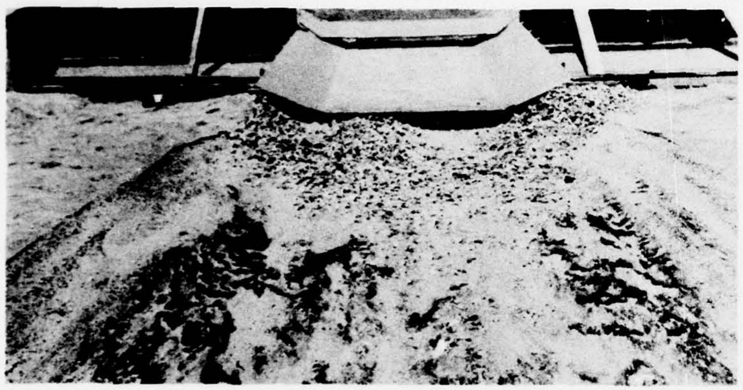
Looking downstream

Looking upstream

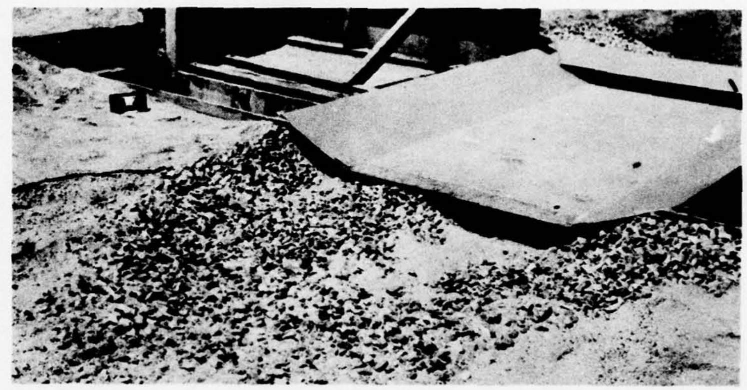


Closeup view from right side

Photo 74. Original design, scour after hydrograph run; peak discharge 29,900 cfs



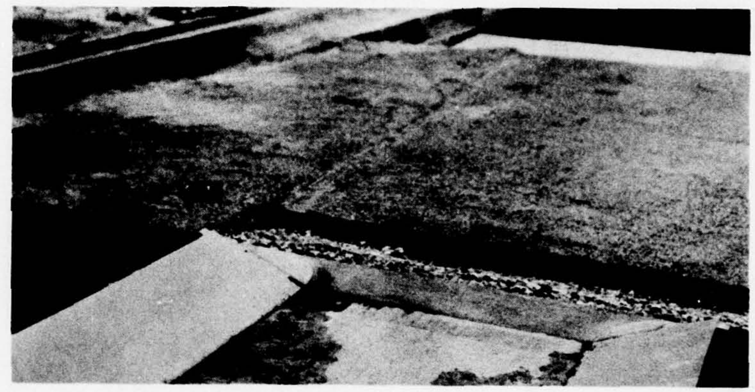
Looking upstream



Looking upstream from left side

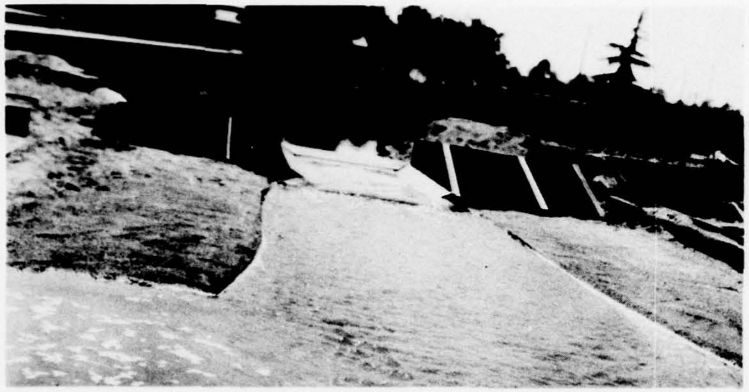
Photo 75. Original design, scour after 1/2-hr run; discharge 51,000 cfs

Looking upstream

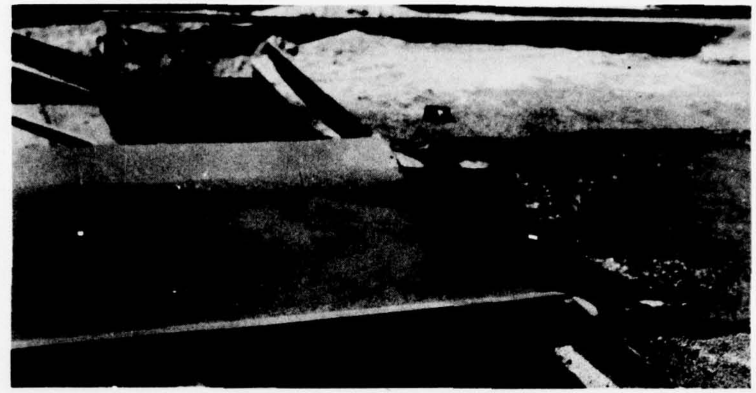


Looking downstream

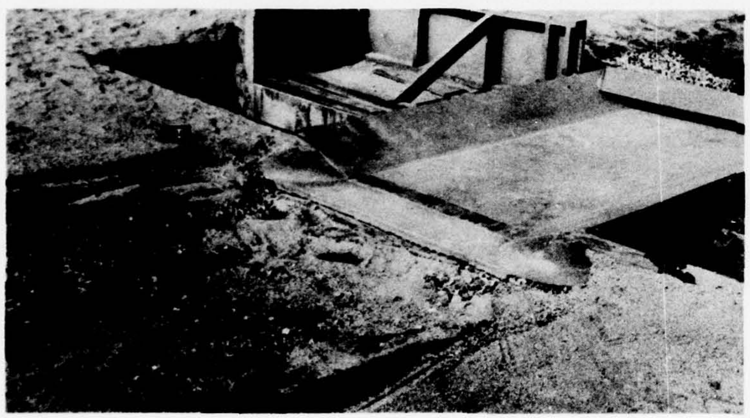
Photo 76. Final design, 1/2-in. rock in place



Looking upstream



View from right side

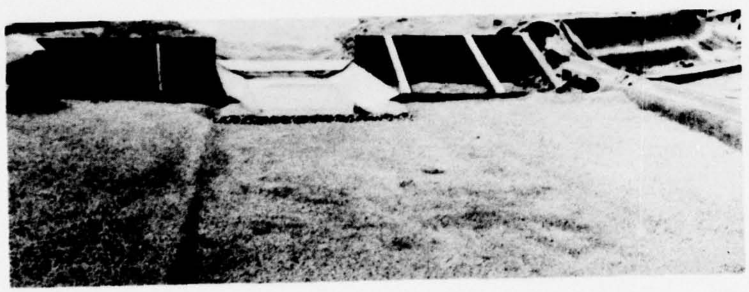


Looking upstream from left side

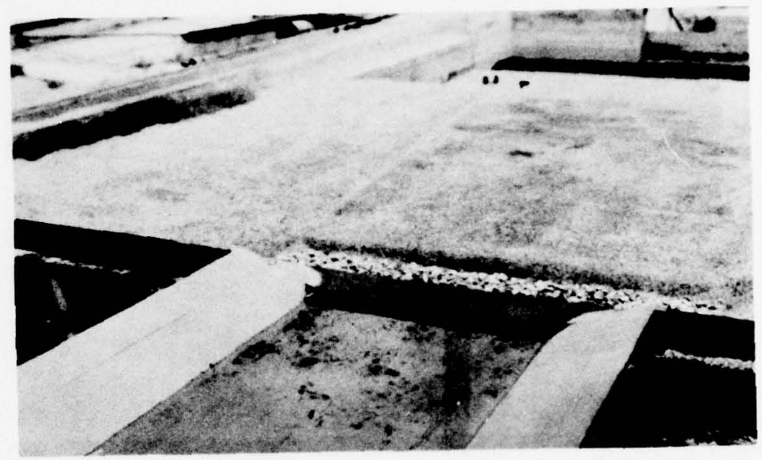
Looking upstream from right side

Scour after hydrograph run

Photo 77. Final design, test with 1/2-in. rock protection, peak discharge 29,900 cfs

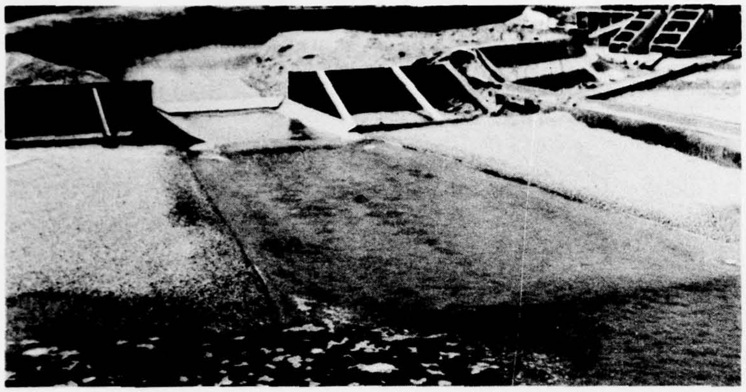


Looking upstream

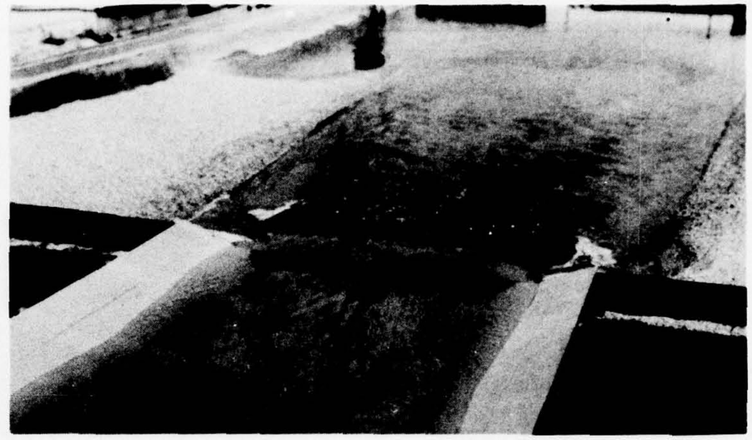


Looking downstream

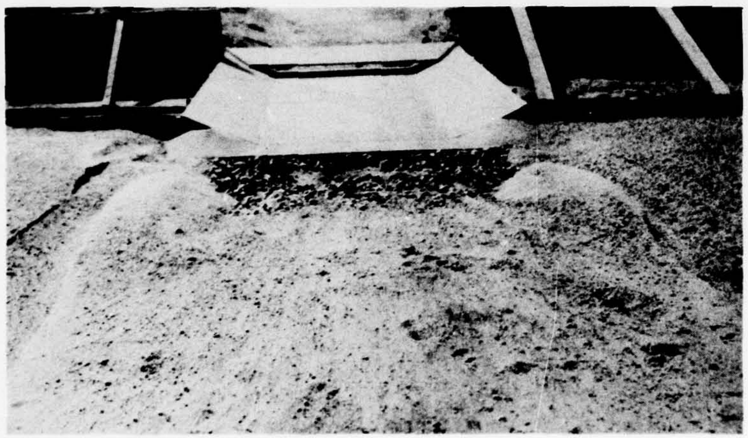
Photo 78. Final design, 1-in. rock in place



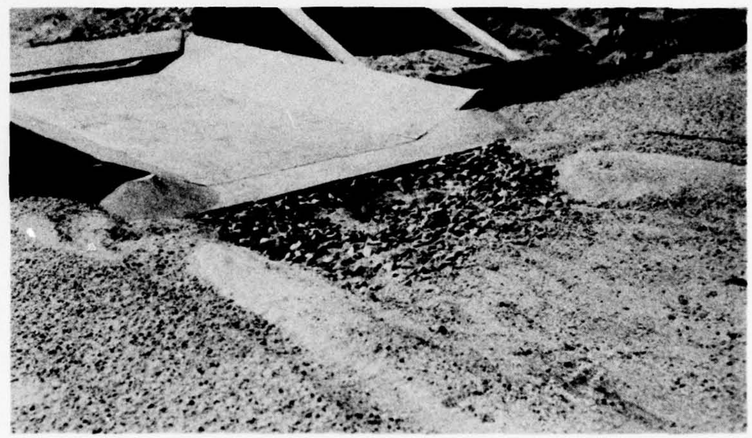
Looking upstream



Looking downstream



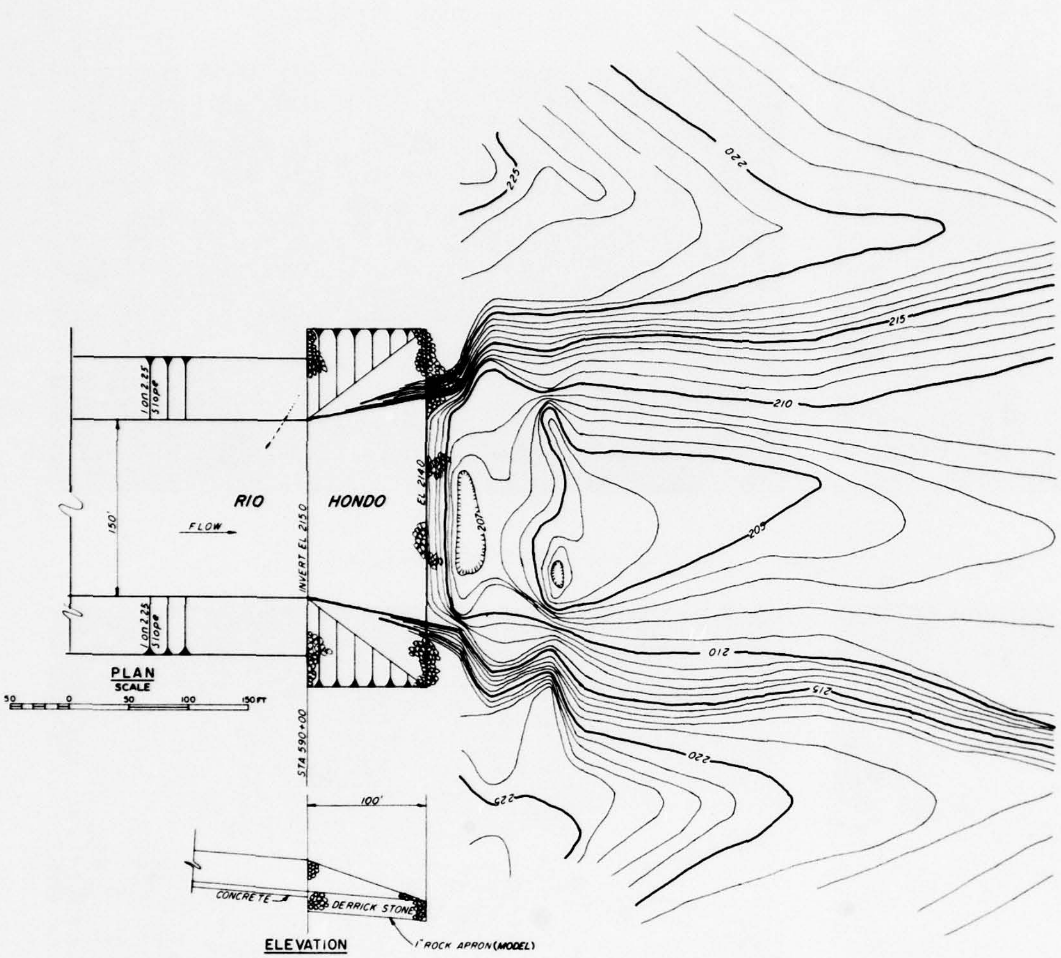
Looking upstream



Looking upstream from right side

Scour after 1/2-hr run

Photo 79. Final design, test with 1-in. rock protection, discharge 51,000 cfs



RIO HONDO INLET INTO WHITTIER NARROWS BASIN

SCOUR PATTERN

ORIGINAL DESIGN

HYDROGRAPH RUN-PEAK 29,900 CFS

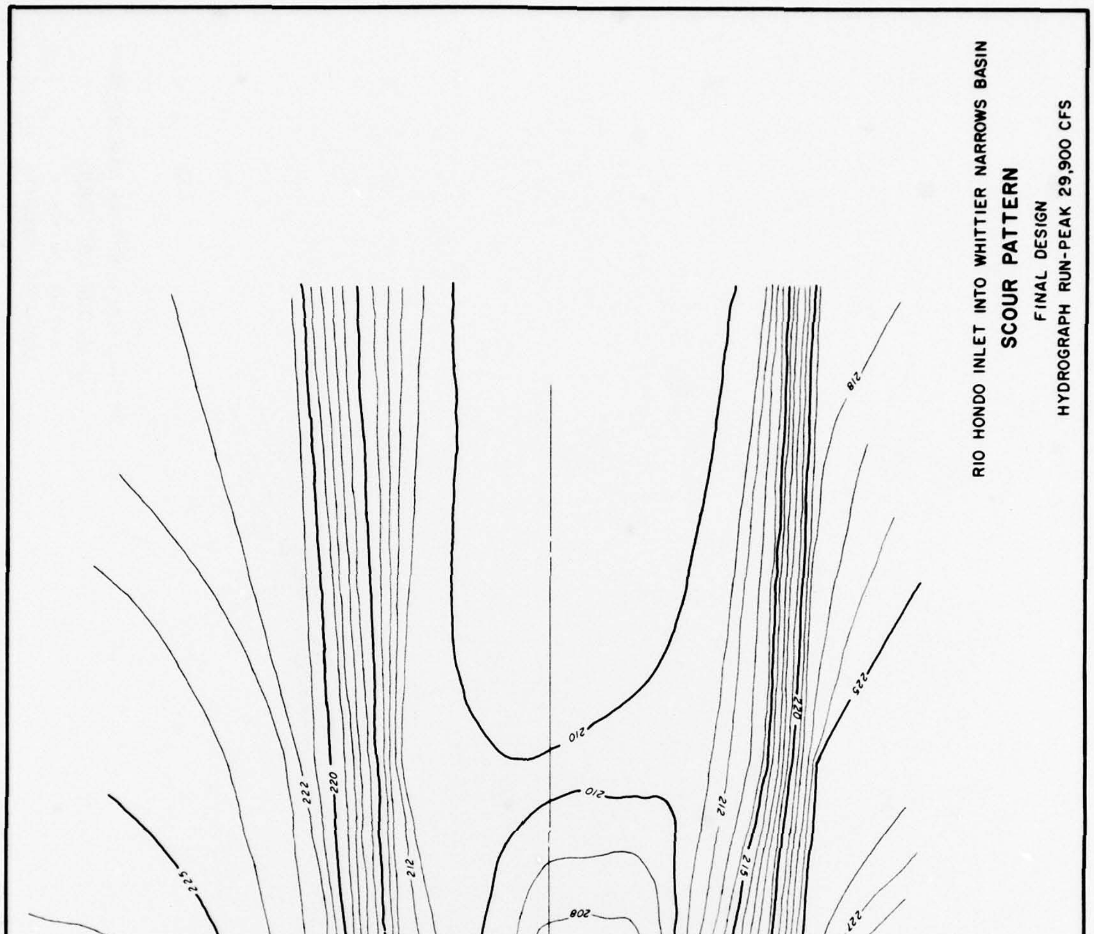
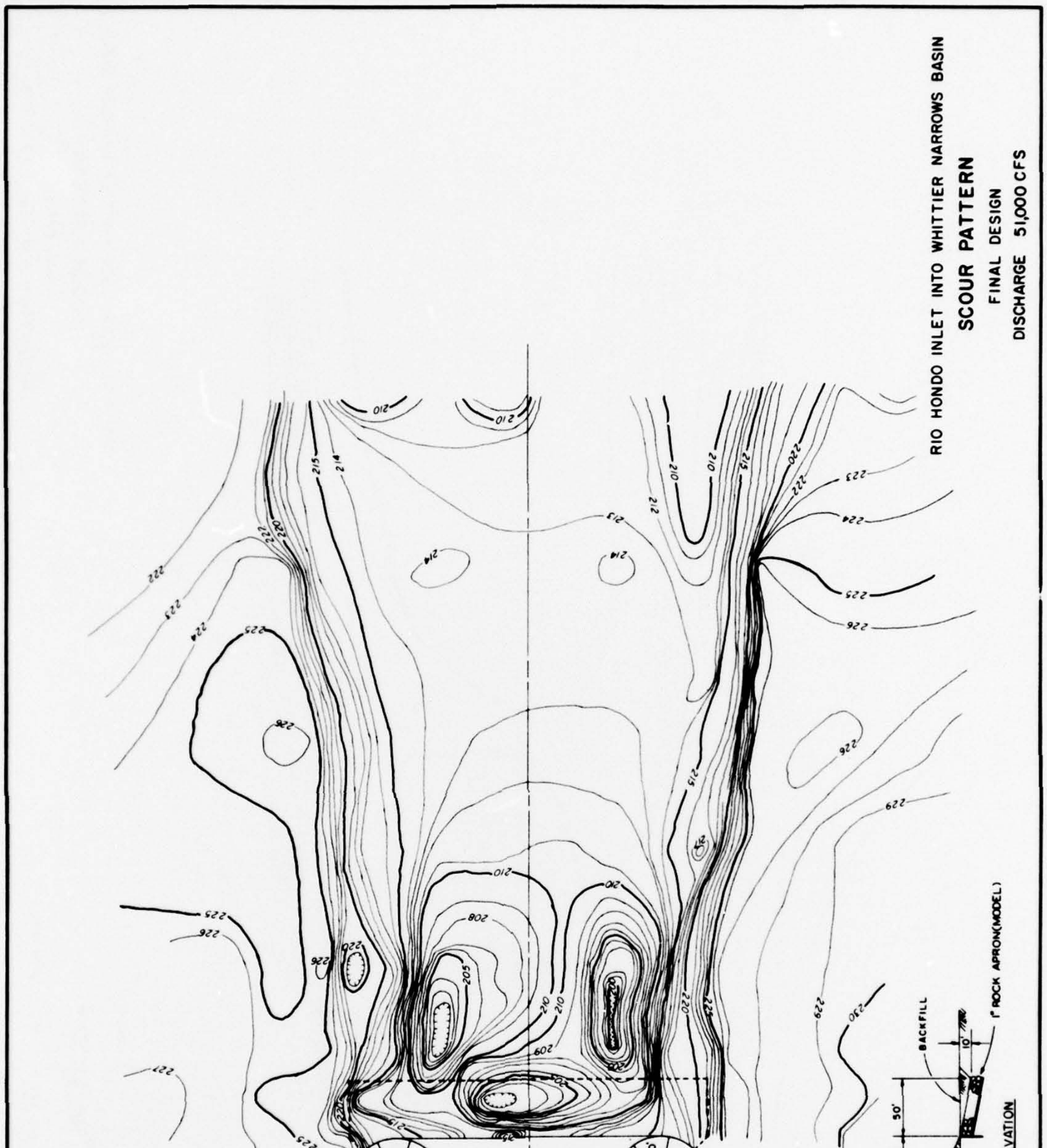

RIO HONDO INLET INTO WHITTIER NARROWS BASIN
SCOUR PATTERN
FINAL DESIGN
HYDROGRAPH RUN-PEAK 29,900 CFS
RIO HONDO
Flow
PLAN
SCALE
ELEVATION
CONCRETE
BACKFILL
1/2" ROCK APRON (MODEL)
STA. 590+00
INVERT EL. 215.50
150'
1 ON 2.25
R=40'
40'
50'
10'
195
200
205
208
210
212
215
218
220
222
225

PLATE 65


RIO HONDO INLET INTO WHITTIER NARROWS BASIN
SCOUR PATTERN
FINAL DESIGN
DISCHARGE 51,000 CFS
RIO HONDO
FLOW
STA 590+00
EL 215.50
PLAN
SCALE
ELEVATION
CONCRETE
BACKFILL
1' ROCK APRON (MODEL)

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

United States. Army. Corps of Engineers.
  Los Angeles District.
  Whittier Narrows Flood Control Basin, Los Angeles County Drainage Area, California; hydraulic model investigation / [by Dave A. Barela]. [Vicksburg, Miss. : U. S. Waterways Experiment Station] ; Springfield, Va. : available from National Technical Information Service, 1979.
  xii, 263 p., 66 leaves of plates : ill. ; 27 cm. (Report - U. S. Army Engineer District, Los Angeles ; 2-112)

1. Diversion channels. 2. Drop structures. 3. Flow characteristics. 4. Hydraulic models. 5. Los Angeles County Drainage Area. 6. Outlet works. 7. Spillways. 8. Whittier Narrows Dam. 9. Whittier Narrows Flood-Control Basin. I. Barela, Dave A. II. Series: United States. Army. Corps of Engineers. Los Angeles District. Report ; 2-112.
TC159.L6 no.2-112